CORPS OF ENGINEERS

NEW ENGLAND DIVISION

ALTERNATIVE BREAKWATER STUDY
BLACK ROCK HARBOR
BRIDGEPORT, CONNECTICUT

VOLLMER ASSOCIATES

JASON M. CORTELL AND ASSOCIATES

JUNE 1981

3RD SUBMISSION FINAL 7-21-81

BLACK ROCK HARBOR

BRIDGEPORT, CONNECTICUT

ALTERNATIVE BREAKWATER STUDY

DEPARTMENT OF THE ARMY

NEW ENGLAND DIVISION, CORPS OF ENGINEERS

WALTHAM, MASSACHUSETTS

JUNE 1981

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BLACK ROCK HARBOR

BRIDGEPORT, CONNECTICUT

ALTERNATIVE BREAKWATER STUDY

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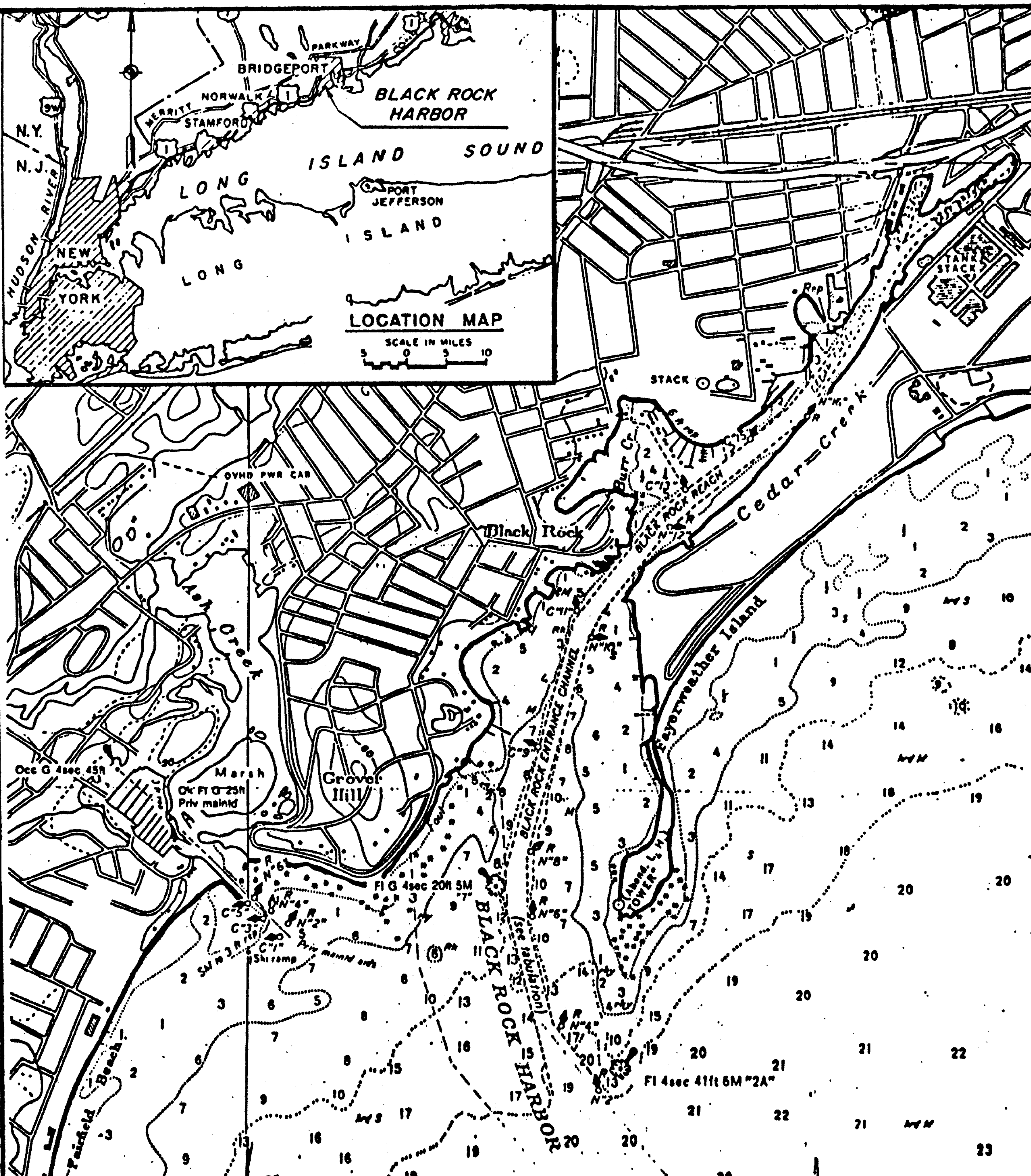
SYLLABUS

Navigation improvements in Black Rock Harbor have been studied for a number of years. In 1958 the Federal government authorized the construction of rock breakwaters on both sides of the entrance to the harbor to protect recreational boats. The breakwaters were never constructed, due to the unavailability of local cost sharing funds. Since that time, a series of studies have been undertaken to determine the economic justification and environmental acceptability of providing navigational improvements within the harbor. A public meeting held in December 1978 documented the substantial storm damage that was inflicted on the large recreational boat fleet and shoreline facilities, due to the lack of protection.

This study, authorized by the Corps of Engineers, New England Division, is an analysis of Alternative Breakwaters at the entrance to Black Rock Harbor. Various types, locations, alignments and dimensions of breakwater structures were evaluated using a developed design storm. The selected breakwater location was chosen to provide the optimum balance between harbor protection and a favorable wave environment at the Breakwater. The selected breakwater type is the concrete composite slope structure.

The findings of this study will be incorporated into the overall feasibility report for Bridgeport Harbor and vicinity.



ALTERNATE BREAKWATER STUDY
BLACK ROCK HARBOR
BRIDGEPORT CONNECTICUT

CORPS OF ENGINEERS, NEW ENGLAND DIVISION

JUNE 1981

SECTION 1

INVENTORY OF EXISTING FACILITIES

The waterfront of the Black Rock Harbor - Cedar Creek has a mixture of public, residential and commercial land uses.

As shown on Figure I-1, the west shore of Black Rock Harbor from the Sound to Burr Creek contains yacht clubs, private homes, marinas and boat yards, while the east shore is vacant city owned park land. The waterfront of this section is utilized entirely for recreational boating. The exact number of recreational boats using this section of waterfront, cannot be determined at this time. The City requires a permit for a boat mooring, but does not enforce the requirement. The yacht clubs and marinas have a limited number of members and moorings, but also serve day-trippers who transport their boats to and from the waterway.

The consensus of the Harbor Master and the yacht club officers and marina operators contacted is that about 1,500 recreational boats use this section of the harbor each season, and the numbers are increasing.

The Cedar Creek section of waterfront inland of Burr Creek is in either public or commercial-industrial ownership, but also contains two recreational boat marinas. Public lands on the west shore contain a sewerage disposal plant and a solid waste processing plant. The solid waste plant is presently closed and neither plant has wharf facilities for marine transportation. On the east shore, the public lands are largely a solid waste landfill with no facilities for marine transportation.

The commercial-industrial ownererships and uses along Cedar-Creek are as follows:

WEST SHORE

Fairfield Scrap Co.

The scrap metal company uses barges to ship processed scrap. Shipments vary widely in number of barges per shipment and frequency, but traffic is reportedly about 100 barges annually. The company expects its barge traffic to increase in the near future. The company is also planning to enter the sand and stone business and will receive material by barge.

Leverity and Hurley (O & G Industries, Inc.)

The company operates an asphalt batch plant, but neither receives nor ships material by marine transportation. However, they are planning to improve their bulkhead and may receive material by barge in the near future.

Connecticut Petroleum Wholesalers (Div.of RH Holcomb)

The company operates a wholesale fuel oil terminal. The oil is received by barge and marine traffic is reportedly about 30 barges annually.

Martin-Marietta Chemical Co. - Refractories Division

The company operates a dry chemical processing plant but neither receives nor ships material by marine transportation.

Southern Connecticut Gas Company

The site contains a 30,000 cf. tank which is used for storage and for the transfer of gas between rail cars and trucks. It does not presently use, or plan to use, marine transporation.

Hi-Ho Fuel Company

The company operates a wholesale fuel oil terminal. The oil is received by barge, but no barge shipments were purchased by the company in 1980. The company purchased their oil from the large shipments received by the other nearby wholesalers. They do, however, expect to purchase 6 to 10 barge shipments this year.

Inland Fuel Terminal

The company operates a wholesale fuel oil terminal. The oil is received by barge and marine traffic is reportedly about 50 barges annually.

Eastern Bag and Paper Company

The company's property is across the end of the east branch of Cedar Creek, but is separated from it by the railroad track. The company, therefore, does not use marine transportation.

EAST SHORE

Fairfield Dock Co.

The company operates a dock building and foundation construction business. The waterfront site is used for material storage and for the loading and off-loading of material, primarily piles and timber, and equipment, including cranes, from barges. Traffic frequency varies widely according to business requirements, but is reportedly as frequent as one barge a week during busy periods.

Bassick Casters (Stewart-Warner Corp.)

This manufacturing plant neither receives nor ships materials or products by marine transportation.

Sikorsky Aircraft

This manufacturing plant neither receives nor ships materials or products by marine transportation.

ALTERNATIVE BREAKWATER STUDY
BLACK ROCK HARBOR
BRIDGEPORT, CONNECTICUT

APPROXIMATE SCALE 1" = 400'

LAND USE INVENTORY
EXHIBIT I-1
JUNE 1981

SECTION 11

AREAS OF REQUIRED PROTECTION

The section of Black Rock Harbor seaward of Burr Creek is the area which is most vulnerable to high water and storm wave damage due to the shape and orientation of the waterway. The waterfront land uses in this area are nearly all small boat related which increases the area's susceptibility to storm damage.

Upstream of Burr Creek, the waterway narrows and is protected by the headland projecting from the West Shore. The land use in this area is mainly industrial with the waterfront protected by bulkheads. Water use is by larger and more substantial vessels less susceptible to wave damage.

The boat clubs and marinas shown in Exhibit I-1 were contacted for information concerning the extent of damage in previous years. Their letters and data are contained in the appendix of the report. A summary of their damages is as follows:

BLACK ROCK YACHT CLUB -	1980	$20,000.-	PROPERTY DAMAGE
		90,000.-	TWO VESSELS DESTROYED
	1979	300,000.-	NINE VESSELS DESTROYED
	1976	120,000.-	TWO VESSELS DESTROYED MISCELLANEOUS OTHER DAMAGE
YE OLDE DOCK MARINA -	1948-1980	$75,000.-	PROPERTY DAMAGE
		100,000.-	VESSEL DAMAGE
PRIVATE MARINA (Kaye Williams) -	1969 - 1980	$60,000.-	PROPERTY AND VESSEL DAMAGE
NAVAL VETERANS PORT 5 -			SUBSTANTIAL EROSION DAMAGE AND PARTIAL COLLAPSE OF SEAWALL.
FAYER WEATHER YACHT CLUB -	1980	$45,000.-	SIXTEEN VESSELS DESTROYED AND PROPERTY DAMAGE
	1978	40,000.-	TWELVE VESSELS DESTROYED
BURR CREEK MARINA -	1980	$50,000 -65,000.-	PROPERTY DAMAGE

SECTION III

DESIGN STORM DETERMINATION

General Description

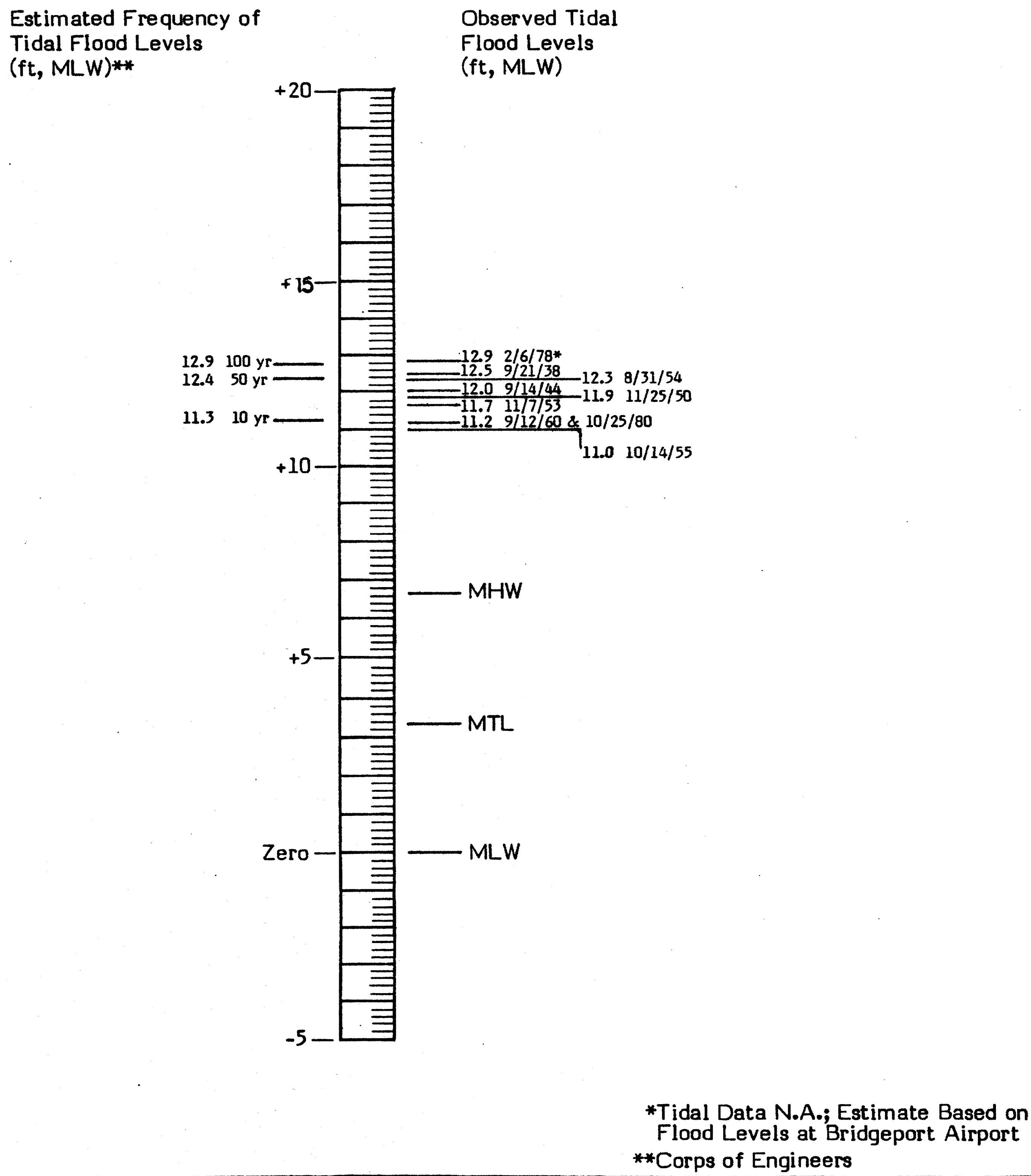
Critical design storm conditions at Black Rock Harbor have been selected to provide coincidence of very high water levels and fully developed wave heights. The general pattern of this storm is largely determined by the southerly orientation of the Harbor mouth and the location of the Harbor on the west side of the north shore of Long Island Sound. Because of the shape and orientation of Long Island Sound, maximum water levels are associated with easterly winds, but southerly winds yield the largest storm waves. For this reason, the design storm is assumed to be a two phase event. First, there will be a period of strong northeast to east winds, causing the maximum static water level in the vicinity of the Harbor. Subsequently, the winds will shift to the south, causing substantial storm waves to enter the Harbor.

Of the large storms or hurricanes which have produced extreme high water levels at Bridgeport in the last 42 years, three coincide with the general design storm criteria for Black Rock Harbor. All of these events were storms, rather than hurricanes. During these events, the central pressures of the storm passed to the west of Bridgeport producing strong easterly winds followed by more southerly. The largest of these storms occurred on November 25, 1950. Easterly winds in excess of 50 knots produced a storm tide recorded at Bridgeport of 11.9 ft elevation (above mean low water - MLW). The wind then shifted to the south-southeast and south-southwest with speeds of 40 to 50 knots, producing high waves and considerable destruction along the southern Connecticut shore.

A slightly smaller storm occurred only three years later, on November 7, 1953, when 30 knot winds from the south-southeast produced large waves on a 11.7 ft storm tide at Bridgeport. More recently, on October 25, 1980, easterly winds of 40 to 47 knots produced a 11.2 storm tide at Bridgeport. These winds were immediately followed by winds from the south-southeast of 30 knots, followed by winds from the southwest of 42 knots. The destruction caused by the waves and flooding during this storm are documented in a later section of this report.

Design Water Level

Records of extreme high water levels at Bridgeport have been compiled by the New England Division, U.S. Army Corps of Engineers (NEDCOE) and the National Weather Service climatologist at Sikorsky Airport in Bridgeport, CT. A composite of these



Alternative Breakwater Study
BLACK ROCK HARBOR ENTRANCE
Bridgeport, Connecticut

COMPARATIVE STORM SURGE DATA
TIDAL DATUM PLANES AND
PAST FLOOD ELEVATIONS
EXHIBIT III-1

records is presented in Exhibit 111-1, along with estimated frequencies of high water levels tabulated by NEDCOE (March, 1979). The local record of high water levels was considered to be of sufficient accuracy and duration for estimation of design water levels.

For preliminary breakwater design purposes, a storm tidal flood stage at 13.0 ft elevation above mean low water was used. This elevation provides conservative design conditions which give maximum wave protection in the harbor. The design water level approximates that with a 100 year frequency, as estimated by the NEDCOE. In addition, a tidal flood level within the past three years (that of February 6, 1978) may have reached an elevation close to the design water level (Exhibit III-1). It should be noted that this water level is not based on the normal National Ocean Survey tidal data, which are not available for February, 1980, rather, it is an estimate based on flood elevations at the Bridgeport Airport on Stratford Point.

Deepwater Waves

The primary collection of wind data for wave forecasting purposes was taken from the U.S. Naval Weather Service Command "Summary of Synoptic Meteorological Observations" (SSMO). These data were available for a 60 year period of record. Local wind data from the Sikorsky Airport National Weather Service station were available for a 12 year period of record and provide a means of comparing the longer SSMO record with local conditions.

The possible direction of significant storm wave attack at Black Rock Harbor is limited not only by the southerly orientation of the Harbor mouth, but also by the wave protection offered by the shoals and reefs off Pine Creek Point and Shoal Point to the southwest and Stratford Point to the east. In addition, selection of a design wind speed is limited by the small possible fetch for winds crossing Long Island Sound. Reference to standard wave forecasting charts in the Shore Protection Manual will confirm that wave generation in high winds from the southeast to southwest will always be fetch-limited for waves approaching Black Rock Harbor. This factor limits the necessary wind duration for maximum wave development to approximately 2 hours; for 75 percent of maximum possible development, the duration is 1-1/4 hours.

The short wind durations necessary for maximum wave development increase the probable frequency of wind conditions producing design wave conditions at Black Rock Harbor relative to open ocean locations. At the same time, short durations limit the accuracy of the 3 hour wind observations tabulated at the Bridgeport Airport for the purpose of estimating critical design waves.

The relevant wind data from the SSMO and the Bridgeport Airport are presented in Exhibit 111-2. Agreement between the two records is limited by the level of resolution of the data tabulation in the SSMO on the one hand and the short period of record at Bridgeport on the other.

EXHIBIT 111-2

FREQUENCY OF WINDS GREATER THAN 22 KNOTS:
SOUTHEAST TO SOUTHWEST QUADRANT

Wind Speed (Knots)

	SSMO			NWS		
Direction	22-33	34-47	48+	22-33	34-47	48+
SE	2.0	*	*	3.0	#	#
SSE	2.0	*	#	1.3	#	#
S	0.7	*	#	0.4	6	#
SSW	1.0	*	#	0.3	12	#
SW	0.7	*	*	0.4	12	#

Return Frequency in Years

SSMO U.S. Naval Weather Service Command, Summary of Synoptic Meterological Observations, New York Area. Period of record = 60 years.

NWS U.S. National Weather Service Bridgeport, Ct. Period of record available = 12 years.

* Percent frequency reported in SSMO is greater than 0.00 and less than 0.05. For the 60 year period of record, one to eight observations are possible with a return frequency of 7.4 to 60 years.

No observations.

A general trend is evident in the 22-33 knot speed class where somewhat greater return frequencies occur in the south to southwest. The 34-47 knot speed class in the SSMO record shows only wind occurrences with a return frequency of 7.4 to 60 years from all directions. This range is supported to the extent possible by the shorter record at Bridgeport which shows return frequencies of 6, 12, and greater than 12 years.

For the purposes of forecasting design wave conditions, it was judged that the 34-47 knot speed class provided winds of sufficient frequency (7.4 to 60 year return) to warrant evaluation and analysis. While the record of winds in this speed class is incomplete, it is important to point out that the SSMO is based on ship observations and tends to underestimate the frequency of high wind conditions which ships would attempt to avoid. When this factor is considered along with the short wind duration necessary for maximum wave development, the probable occurrence of winds in this speed class is of sufficient frequency to warrant consideration. Winds in the highest speed class (+47 knots), on the other hand, appear to be too infrequent for reasonable design considerations. A speed of 42 knots was taken as representative of critical wind conditions within the 34-47 knot speed class, balancing the extreme speed for the class with its average return frequency.

Wave height and period were estimated for winds of 42 knots from the SE, SSE, S, SSW, and SW using the modified SMB method. Although portions of the fetch from these directions are in water that is slightly shallower than strict deepwater conditions, the effect of bottom friction on waves of the size estimated may be assumed to be negligible. Use of the deepwater forecasting technique provides slightly conservative design conditions. The forecast wave heights, periods, and wave lengths are presented in Exhibit 111-3 along with the fetch length from the various directions. All forecast waves are fetch limited, with wind durations necessary for maximum possible development of 1.8 to 2.2 hours.

EXHIBIT 111-3

ESTIMATED DEEPWATER SIGNIFICANT WAVE HEIGHT/PERIOD AND LENGTH

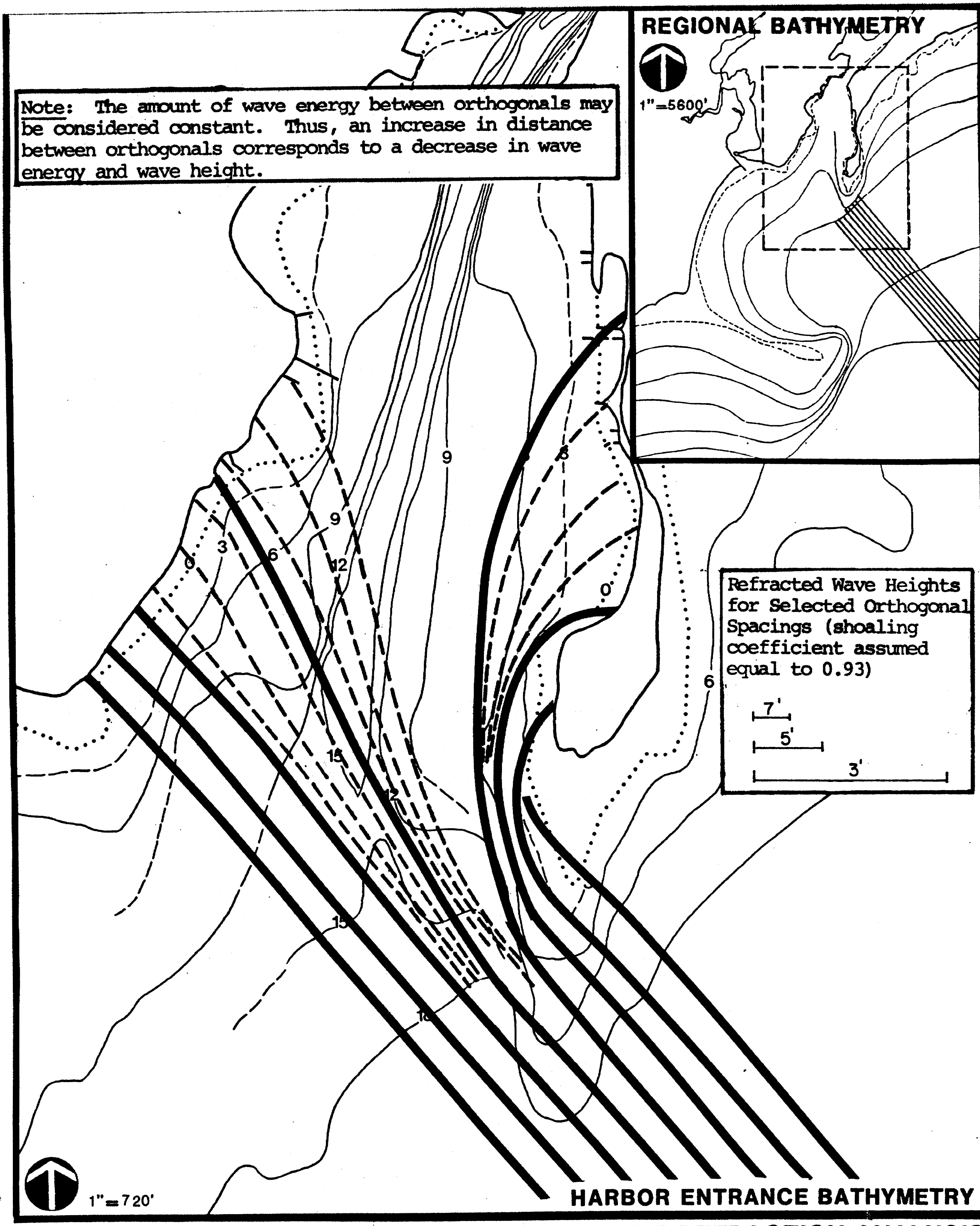
Direction	Fetch Length (nautical miles)	Height (ft)	Period (sec)	Length (ft)
SE	15	8.0	6.0	184
SSE	11	7.0	5.6	161
S	14	7.9	5.9	178
SSW	12	7.2	5.7	166
SW	15	8.0	6.0	184

Wind Speed: 42 Knots
Wave Length: $L_o = 5.12T^2$

Wave Refraction

Refraction of the deepwater waves presented in Exhibit 111-3 was performed using standard graphical techniques. A deepwater orthogonal spacing of 178 ft was chosen to approximate one wavelength. Bathymetric contours of 0, 6, 12, 18, 24, 30, and 60 ft (below MLW) were used. Exhibits 111-4 through 111-8 present the refraction diagrams for the five directions and related wave parameters used. Exhibit III-9 presents the refraction coefficients (K_R) and wave heights obtained at the Harbor entrance. It may be seen that the highest K_R values were obtained for waves arriving from the SSE, with wave energy concentrated on the western side of the Harbor entrance, the east side being slightly in the lee of Fayerweather Island. Waves arriving from the SE have K_R values slightly less than those from the SSE due to their more oblique approach to the Harbor entrance. Wave energy at the Harbor entrance is approximately comparable between the two directions, however, as the deepwater waves arriving from the SE are slightly larger. Waves arriving from the S to SW have lower K_R values owing to a greater refraction of wave energy in passing over the shoals and reef off of Shoal Point.

Refraction of the deepwater waves into the Harbor was calculated using bathymetric contours at 0, 3, 6, 9 12, 15 ft (below MLW). Additional orthogonals were placed between the deepwater orthogonals at the Harbor entrance to provide a more detailed estimate of the refraction patterns within the Harbor. The results of these refraction analyses are also presented in Exhibits 111-4 through 111-8. It may be seen in these Exhibits that the main areas of relatively high wave energy within the Harbor are along the outer western shore under southeast, south-southeast and south-southwest wave attack along parts of the Fayerweather Island seawall under south-southwest waves. Refraction coefficients at the head of the Harbor are uniformly low under all wave directions.



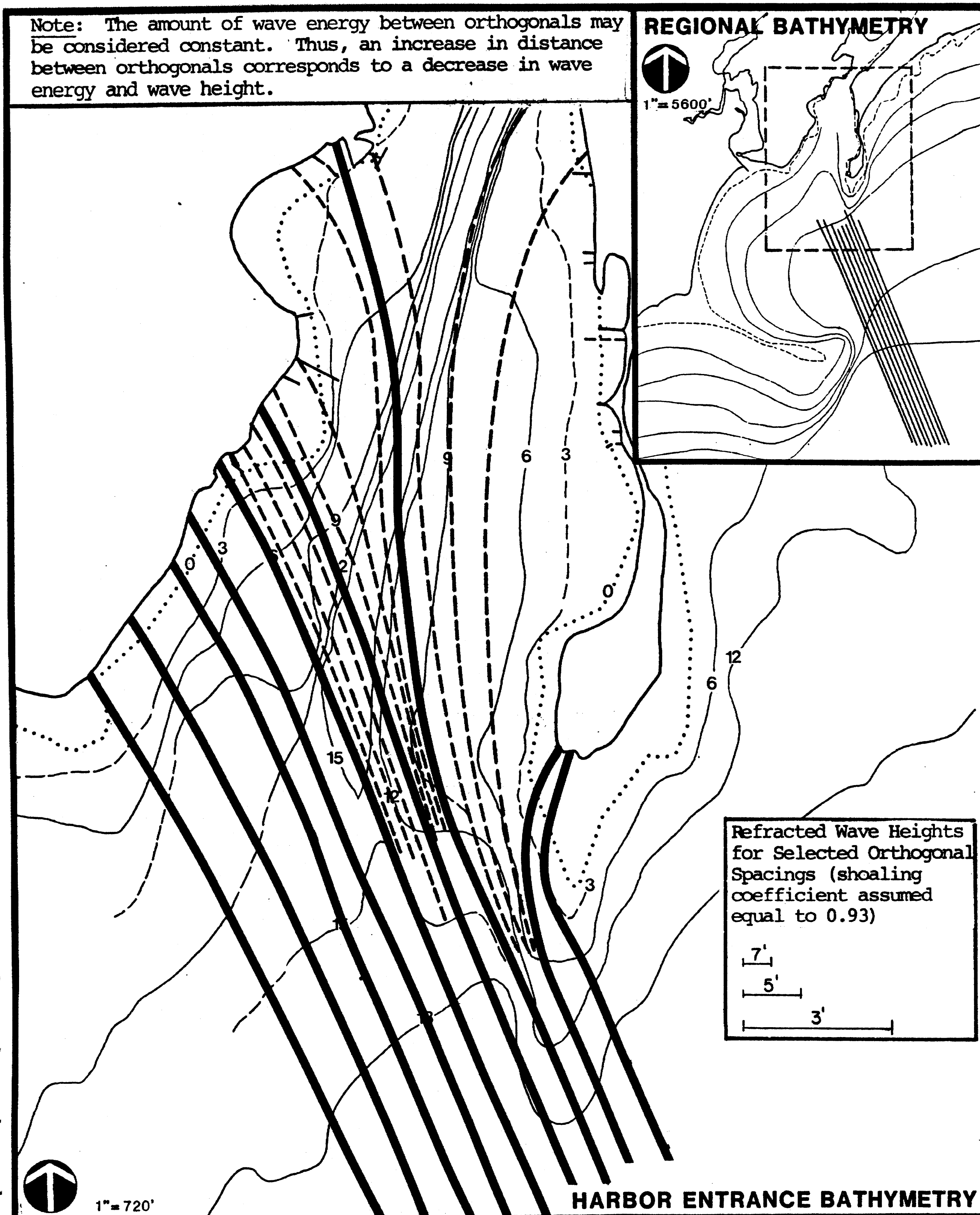
Alternative Breakwater Study
BLACK ROCK HARBOR ENTRANCE
Bridgeport, Connecticut

WAVE REFRACTION ANALYSIS
Southeast Wave Approach

- Deepwater Wave Orthogonals
- Inferred Orthogonals
- —6— Contours-Feet Below MLW

EXHIBIT III-4

Note: The amount of wave energy between orthogonals may be considered constant. Thus, an increase in distance between orthogonals corresponds to a decrease in wave energy and wave height.

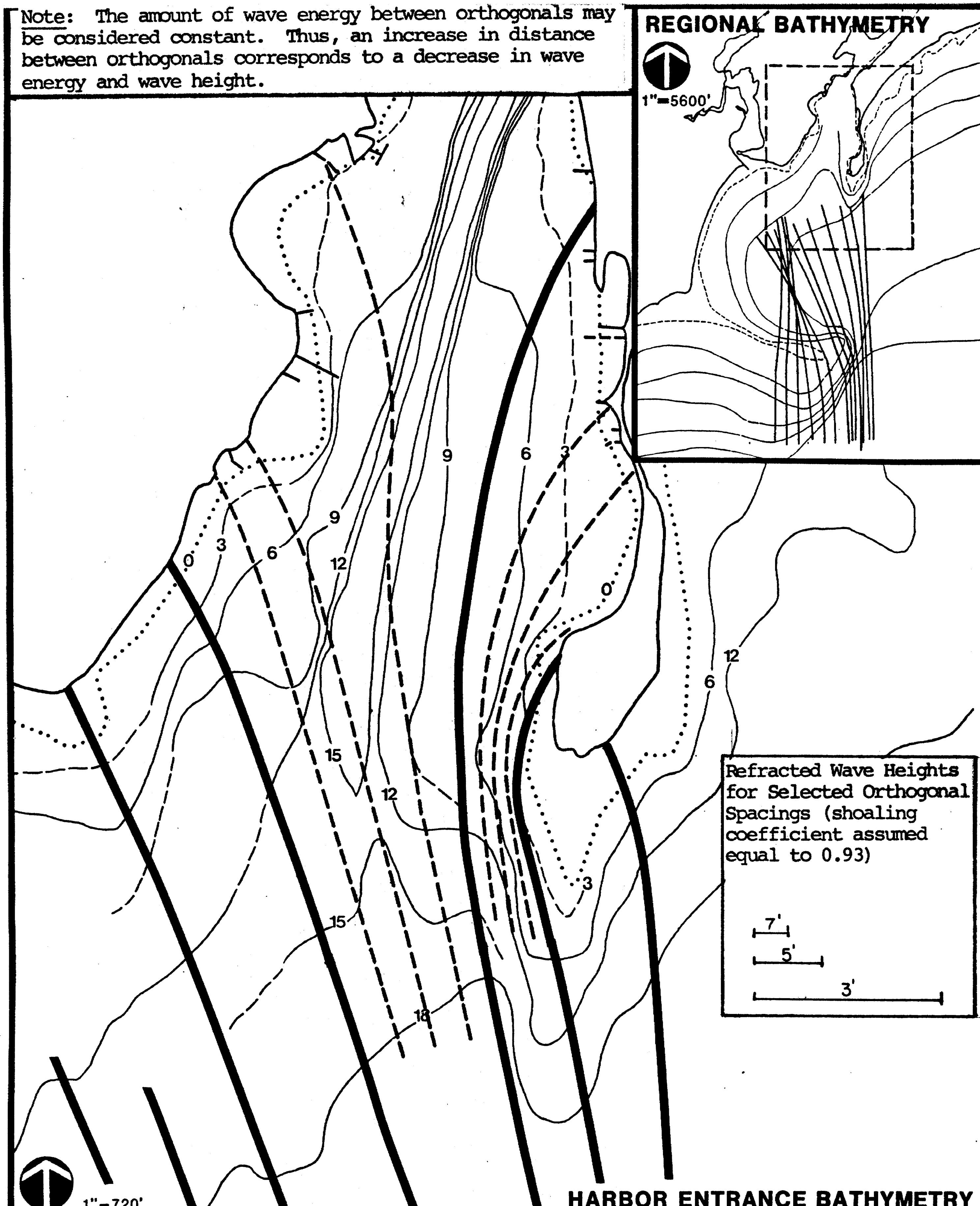


Alternative Breakwater Study
BLACK ROCK HARBOR ENTRANCE
Bridgeport, Connecticut

WAVE REFRACTION ANALYSIS
South-Southeast Wave Approach

Deepwater Wave Orthogonals
Inferred Orthogonals
–6– Contours-Feet Below MLW

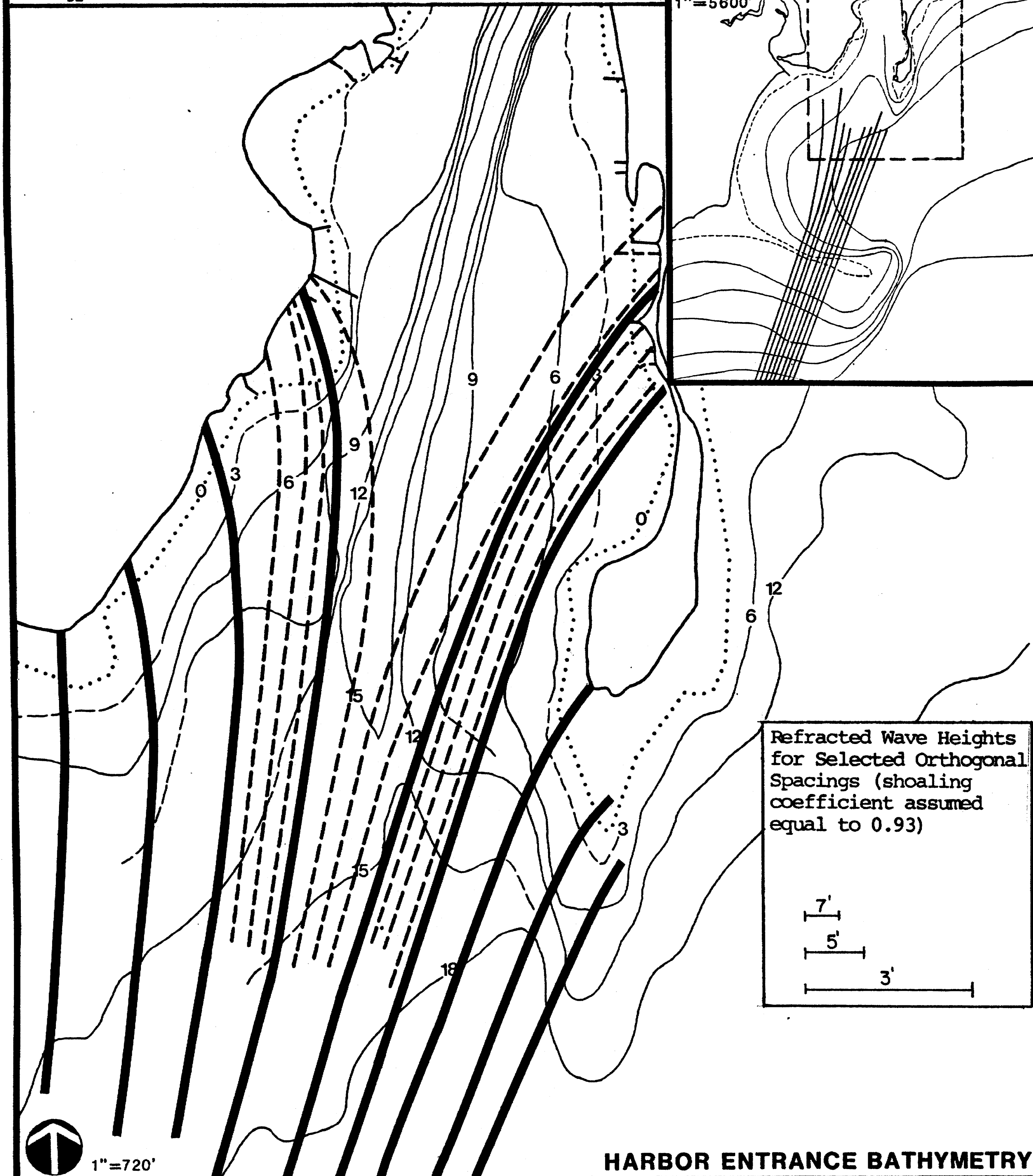
EXHIBIT III-5



Alternative Breakwater Study
BLACK ROCK HARBOR ENTRANCE
Bridgeport, Connecticut

WAVE REFRACTION ANALYSIS
South Wave Approach
Deepwater Wave Orthogonals
Inferred Orthogonals
—6—Contours-Feet Below MLW
EXHIBIT III 6

Note: The amount of wave energy between orthogonals may be considered constant. Thus, an increase in distance between orthogonals corresponds to a decrease in wave energy and wave height.

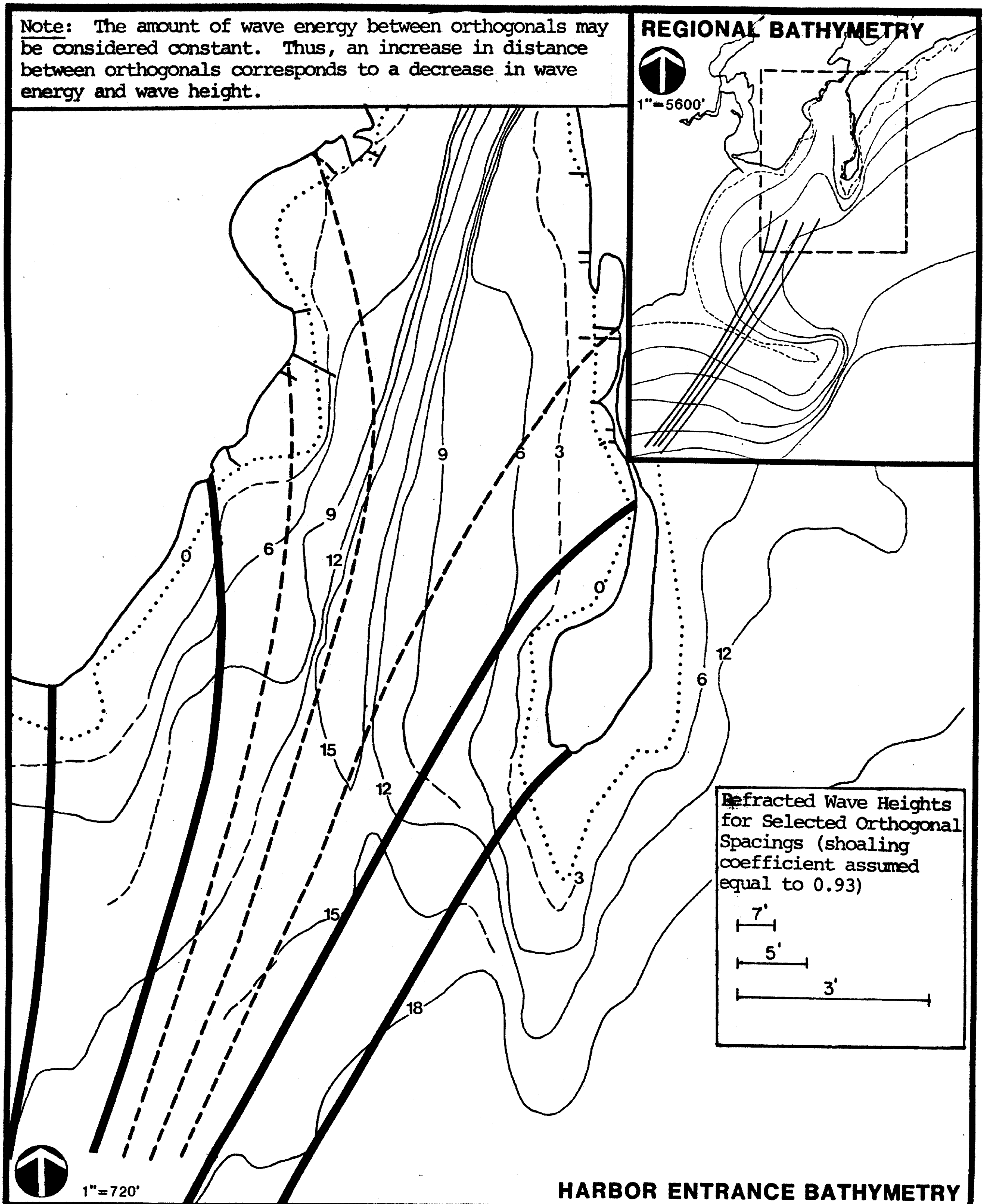


Alternative Breakwater Study
BLACK ROCK HARBOR ENTRANCE
Bridgeport, Connecticut

WAVE REFRACTION ANALYSIS
South-Southwest Wave Approach

Deepwater Wave Orthogonals
Inferred Orthogonals
—6— Contours-Feet Below MLW

EXHIBIT III-7



Alternative Breakwater Study
BLACK ROCK HARBOR ENTRANCE
Bridgeport, Connecticut

WAVE REFRACTION ANALYSIS
Southwest Wave Approach

- Deepwater Wave Orthogonals
- Inferred Orthogonals
- 6 – Contours-Feet Below MLW

EXHIBIT III - 9

AVERAGE DESIGN STORM WAVE HEIGHT, SHOALING AND REFRACTION COEFFICIENTS AT ENTRANCE OF BLACK ROCK HARBOR
(42 KNOT WIND; DURATION > 2.2 HOURS)

Direction			West Half of Harbor Entrance		East Half of Harbor Entrance		Average for Harbor Entrance	
	Water Depth (ft)	Shoaling Coefficient K_S	Refraction Coefficient K_R	Wave Height (ft)	Refraction Coefficient K_R	Wave Height (ft)	Refraction Coefficient K_R	Wave Height (ft)
SW	13	0.97	0.36	2.8	0.34	2.6	0.35	2.7
	19	0.93	0.36	2.7	0.34	2.5	0.35	2.6
	25	0.92	0.36	2.7	0.34	2.5	0.35	2.6
SSW	13	0.96	0.54	3.8	0.55	3.8	0.55	3.8
	19	0.92	0.54	3.6	0.55	3.7	0.55	3.6
	25	0.91	0.54	3.6	0.55	3.6	0.55	3.6
S	13	0.97	0.36	2.8	0.48	3.6	0.42	3.2
	19	0.93	0.36	2.7	0.48	3.5	0.42	3.1
	25	0.91	0.36	2.6	0.48	3.5	0.42	3.0
SSE	13	0.95	0.75	5.0	0.52	3.4	0.65	4.3
	19	0.92	0.75	4.8	0.52	3.3	0.65	4.2
	25	0.91	0.75	4.8	0.52	3.3	0.65	4.2
SE	13	0.97	0.54	4.2	0.55	4.3	0.55	4.3
	19	0.93	0.54	4.1	0.55	4.1	0.55	4.1
	25	0.92	0.54	4.0	0.55	4.0	0.55	4.0

SECTION lV

BREAKWATER LOCATIONS

The location of the breakwaters at Black Rock Harbor can influence both the cost of construction and the degree of protection offered to the Harbor. For this reason, several alternative locations were considered as part of this Alternative Breakwater Study. A selection from among the alternatives was made based primarily on protection offered to the Harbor, with consideration of cost used to select among alternatives offering similar protection.

Alternative Locations

An examination of Exhibits lll-4 through lll-8 indicates that wave refraction near the Harbor will produce the most substantial waves along the western shore. Much of the eastern shore is protected for many wave directions by the Fayerweather headland. Thus, the aim of any breakwater location will be to provide protection to the inner Harbor and anchorage locations, with particular emphasis on intercepting waves bound for the western shore of the inner Harbor

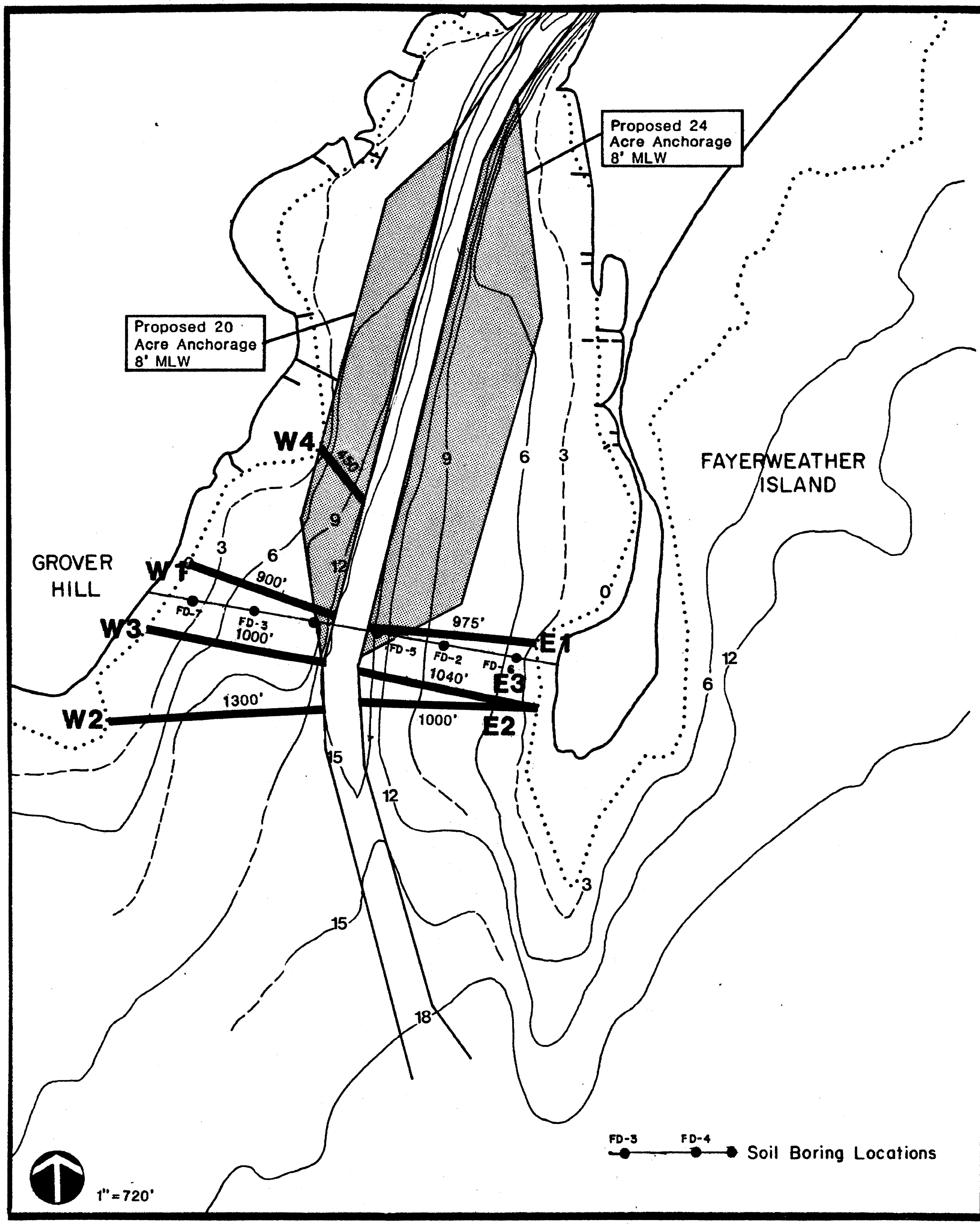
Exhibit lV-1 shows the range of alternative breakwater locations examined in this study. Each offers advantages and disadvantages.

Location Alternative EI-WI is approximately that chosen by the Corps in their original study. It offers protection for most of the Harbor and nearly all of the proposed anchorage areas with relatively short breakwaters. Assuming that both halves of the breakwater will extend from the Mean Low Water line to the edge of the entrance channel, the eastern breakwater (EI) will be 975 ft in length and the western (WI) will be 900 ft long.

Location Alternative E2-W2 is located at the outer end of the Harbor. Such a location offers protection to the entire inner Harbor,the outer Harbor, and the entire western shoreline. A substantial increase in length, however, is required. At this location, the eastern half of the breakwater (E2) would be 1,000 ft long and the western half (W2) 1,300 ft.

Intermediate between these two is Alternative E3-W3. It offers both protection and total breakwater length between those of the El-Wl and E2-W2 Alternatives. The length of (E3) would be 1040 ft and (W3) would be 1000 ft.

Finally,Alternative location W4 was investigated. It could be paired with any of the eastern locations to offer protection only for the inner Harbor. It is the shortest of the western breakwaters, at 450 ft, but leaves much of the western shore of the outer Harbor exposed.



Alternative Breakwater Study
BLACK ROCK HARBOR ENTRANCE
Bridgeport, Connecticut

ALTERNATIVE BREAKWATER LOCATIONS

EXHIBIT IV-1

Comparison of Alternatives

Because of the differences in wave size and approach direction for the eastern and western locations, each group was considered separately. In this fashion, combinations of breakwaters with both aligned gaps and staggered gaps are possible.

Among the western locations, W4 was eliminated from consideration immediately. While it does offer some protection to the inner Harbor and is the shortest of the western alternatives, many of the shore facilities most in need of protection are seaward of this location. W4 would offer construction cost advantages, but only at the penalty of greatly reduced protection of the western shore and the loss of most of the proposed western anchorage.

Considerations of potential cost and needed protection also led to the elimination of Alternatives W_2 and W3. The shoreline south of location W1 is now well protected by seawalls and there are no anchorage or dock facilities where damage to moored craft could occur. Thus, the additional cost associated with the greater length and deeper water depths for W_2 and W_3 cannot be balanced against any increase in protection.

Among the eastern breakwater locations, the choice is not as clear cut. All three locations offer substantially the same protection to the Harbor and are approximately the same length. E3, however, can be eliminated from consideration, since it is functionally equivalent to location E2 and is slightly longer.

Placement of the breakwaters at any of the alternative locations is not expected to influence littoral drift patterns in the area. Although no local data on littoral drift are available, the absence of sand in the harbor mouth geologic section suggests that little or no sand transport across the harbor mouth occurs. Placement of the breakwaters in the harbor mouth will result in a concentration of terrestrial and tidal discharge through the harbor mouth. This factor will result in an increase in tidal currents in the boating channel at the center of the harbor mouth as well as at the shoreward breakwater gaps, if these are included in development plans. The increase in tidal currents should be relatively greater for those breakwater location alternatives which have a narrow, rather than staggered gap at their center. No tidal current data are presently available for Black Rock Harbor and estimation of tidal currents with project development will require a study of the tidal prism exchange and terrestrial discharge in the harbor. Such a study should be conducted prior to any final design for harbor protection.

Recommended Locations

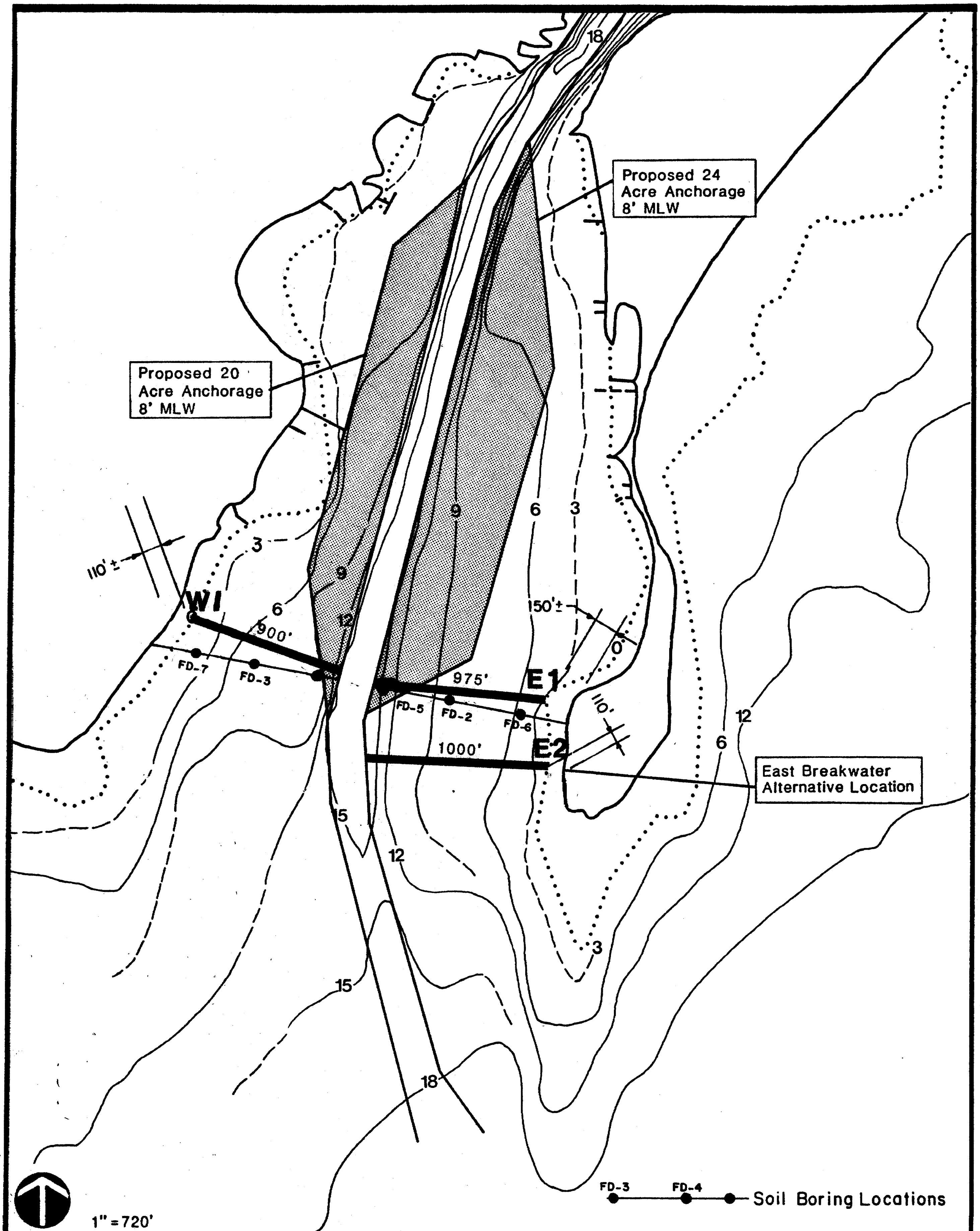
The recommended breakwater locations are presented in Exhibit IV-2. These locations offer the advantage of minimum length while providing a balance between minimizing the wave energy at the breakwaters and maximizing the protection offered to the Harbor. The recommended locations are outboard of all existing Harbor facilities requiring protection and protect virtually all of the proposed anchorage within the Harbor. The west breakwater location (W1) is also oriented at a slight angle to the dominant waves arriving from the southeast to south-southeast decreasing the wave energy per unit length along the breakwater. A higher angle to these dominant waves was also considered for the west breakwater, but is not practical given the increase in length necessary to decrease significantly the wave energy along the breakwater into a more energetic wave environment in order to match the western one.

The east breakwater location (E-1) also offers a minimum length while providing protection to all of the Harbor facilities. This location is also recessed as far as possible behind Fayerweather Island to provide maximum reduction of wave heights at the breakwater. The W1-E1 breakwater location offers the further advantage that subsurface investigations have already been completed along a transect close to the breakwater locations.

An alternative location for the east breakwater (E-2) was also considered. This alternative location provides added protection from southeast and south-southeast waves by eliminating any gap through which waves could pass directly. This location also offers the option of providing additional anchorage at some time in the near future.

Evaluation of this alternative from the standpoint of its potential added wave protection, however, suggests that it may not be the optimum plan. It was found that breaking wave heights due to the diffraction of waves through the gap of either breakwater combination were consistently less than those which would be produced within the Harbor by the same design storm winds. Consequently, the placement of the east breakwater further outboard would not provide any appreciable difference in the wave activity within the Harbor.

The breakwater length, wave evergy, and wave protection criteria mentioned above apply equally to all breakwater types examined in this report. Based on these considerations, the same breakwater locations are recommended for all types of fixed and floating structures.



Alternative Breakwater Study
BLACK ROCK HARBOR ENTRANCE
Bridgeport, Connecticut

RECOMMENDED BREAKWATER LOCATIONS
EXHIBIT IV-2

The breakwater locations are placed to provide a 200 ft boating channel gap as proposed by NEDCOE. The shoreward end of the breakwaters is located at the MLW contour. The gap between the shoreward end of the breakwater and the land provides additional tidal prism exchange for the Harbor with a negligible loss of wave protection. The lengths of the proposed breakwaters are 900 ft, 975 ft, and 1000 ft for the West, East 1 and East 2 alternatives, respectively.

SECTION V

BREAKWATER TYPES AND DIMENSIONS

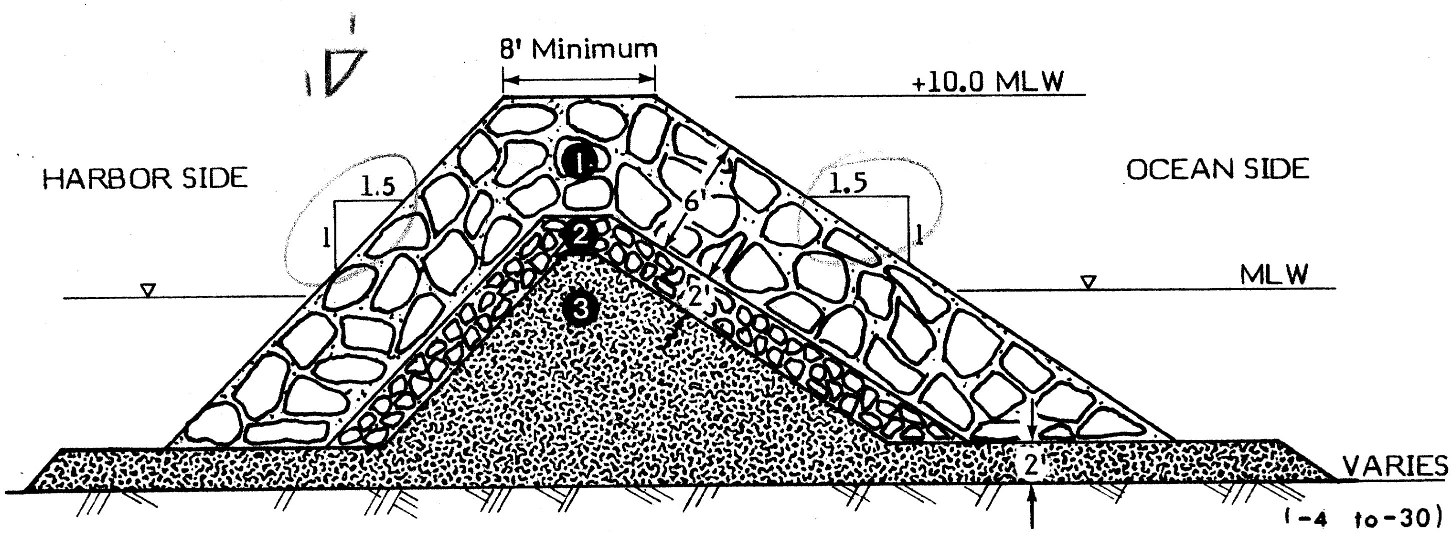
Following the consideration of alternative locations for the breakwaters at Black Rock Harbor, the next step in the process is the consideration of alternative designs for the breakwaters. After a review of the design storm information and refraction diagrams it was apparent that several forms of breakwater have the potential to provide protection for small craft in the Harbor. The most promising of these are described in detail below.

Breakwater Designs

As part of this study, five major types of breakwater were investigated. These were rubble mound structures, sheet pile cellular structures, thin walled sheet pile structures, composite structures, and floating breakwaters. All had some initial promise, although the analysis described below led to the elimination of the floating breakwaters and one of the composite structures from further consideration.

Rubble Mound Breakwaters

The first type considered was the traditional rubble mound breakwater. This is a time-tested design and has been used successfully at many locations in the area, including Bridgeport Harbor immediately to the east of Black Rock Harbor. Exhibit V-1 shows the details of a conventional rubble mound breakwater designed for the storm wave conditions at Black Rock Harbor. Stone size in the three layers of the structure was selected according to the guidance in the Shore Protection Manual. Armor stone size was selected conservatively for a 6 ft wave and is approximately 2 tons. Either quarry stone or reinforced concrete armor units can be used in the cross-section, depending on relative cost. In either case, crest width of at least 8 ft is required both for stability and for ease of construction. Because wider crests provide substantial improvements in wave reduction, especially for lower heights, a crest width of up to 20 ft is carried forward in this analysis. For all variations of this basic design, crest elevations may vary from as little as 8 ft above MLW to more than 15 ft above MLW. The lower limit was selected by observation of the performance of the Bridgeport breakwaters. The upper limit is based on wave reduction requirements.



TYPICAL CROSS SECTION WITH CREST ELEVATION 10.0' MLW

Scale 1" - 10'

MATERIALS: ❶ 2 Ton Stone Armor OR Concrete Armor Units (3.3')
❷ Filter Layer of 300# to 500# Stone (1.4' - 1.7')
❸ Core and Bedding of 1# to 30# Stone (2.5" - 8")

CREST ELEVATION: Crest Elevation may vary from 8' MLW to >15' MLW

RUBBLE MOUND BREAKWATER

EXHIBIT V-1

Cellular Sheet Pile Breakwater

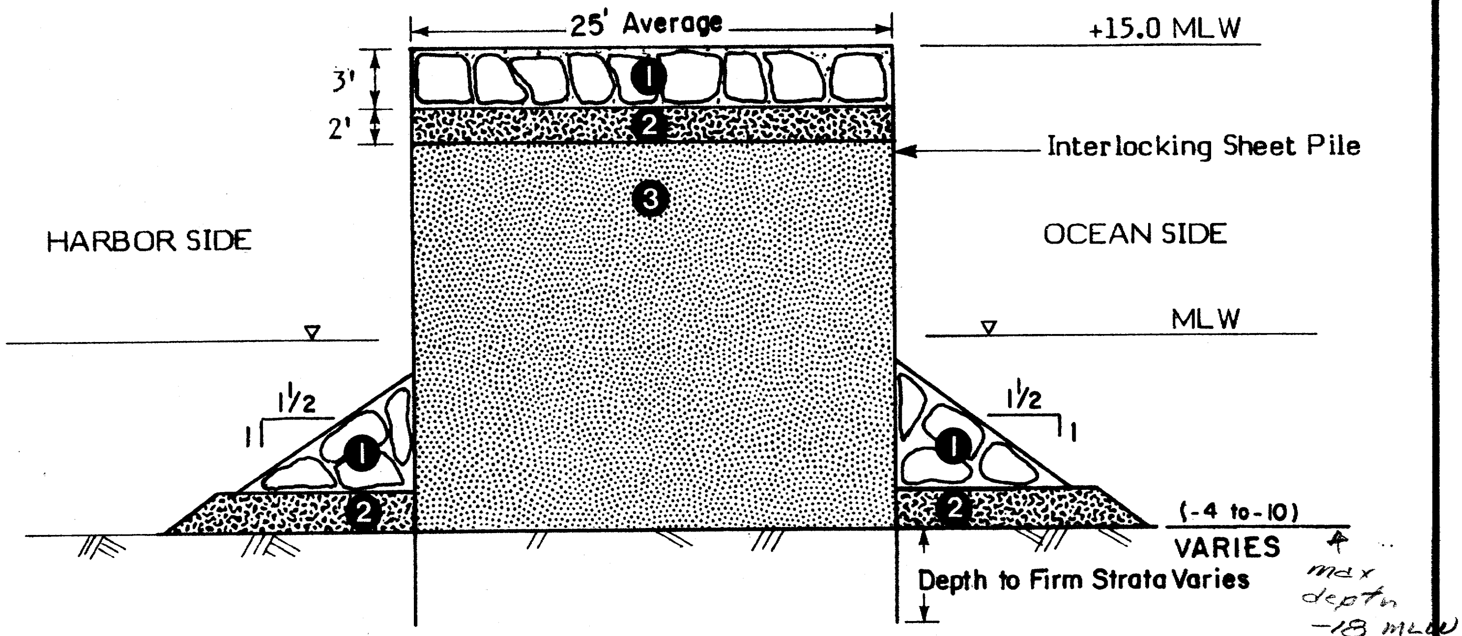
Depending upon cost and availability of suitable rubble material and on foundation conditions, a cellular sheet pile structure could provide protection for the Harbor and be competitively priced. Exhibit V-2 shows a cellular sheet pile structure that could be used successfully at Black Rock Harbor. For this breakwater, armor stone is required only on the top of the cells and in the toe protection, reducing the need for large stone or armor units. Again, a 2-ton stone or armor unit is recommended. A cell diameter of 25 ft was selected so that overturning moments could be resisted easily and for ease in construction with standard sheet pile shapes, such as PS-4 or PS-5 sections. Because of the semi-flexible behavior of such a structure, the Shore Protection Manual suggests a more conservative design to limit overtopping. For this reason, a minimum crest elevation of 15 ft is suggested for this type of structure.

Thin Wall Sheet Pile Breakwater

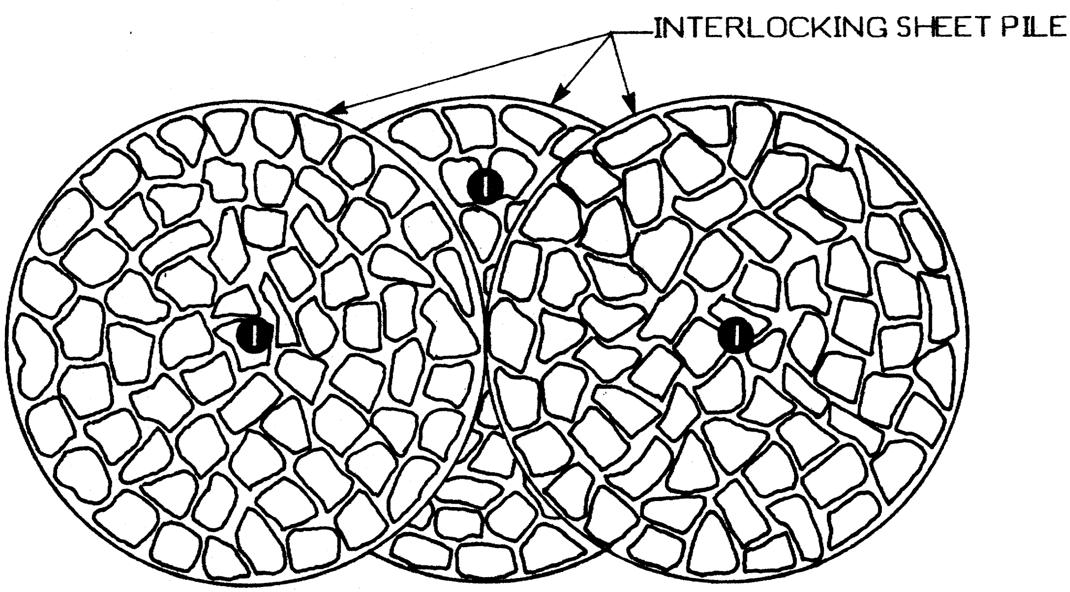
Consideration was also given to a single wall sheet pile breakwater with top bracing and integral back stays for moment resistance. This concept is presented in Exhibit V-3. The design is based on the use of standard PZ-38 sheet pile for both the wall and the back stays, although other sections could be chosen at the time of final design. The top brace shown is a single H-beam. As with all of the fixed structures, toe protection is provided by 2-ton armor stone or concrete armor units. It should be noted that batter piles were also considered for the back stays, but as discussed in Section IX, foundation conditions do not allow the development of sufficient pile bearing capacity for this application. Thus, only the integral back stay arrangement is considered here.

Composite Structures

In the course of this study, two composite structures were considered as possible candidates for the Black Rock Harbor breakwaters. These included a flexible concrete wall supported by piles and a concrete composite slope breakwater.



TYPICAL CROSS-SECTION WITH CREST ELEVATION 15' MLW   Scale 1" = 10'



PLAN   Scale 1" = 10'

MATERIALS: Interlocking Steel Sheet Pile
1 2 Ton Armor Stone,
2 Bedding Stone, 2.5" - 8"
3 Cell Fill Material, Sand to 8" Material

CREST ELEVATION: Minimum 15' MLW will meet deisgn wave criteria

Alternative Breakwater Study
BLACK ROCK HARBOR ENTRANCE
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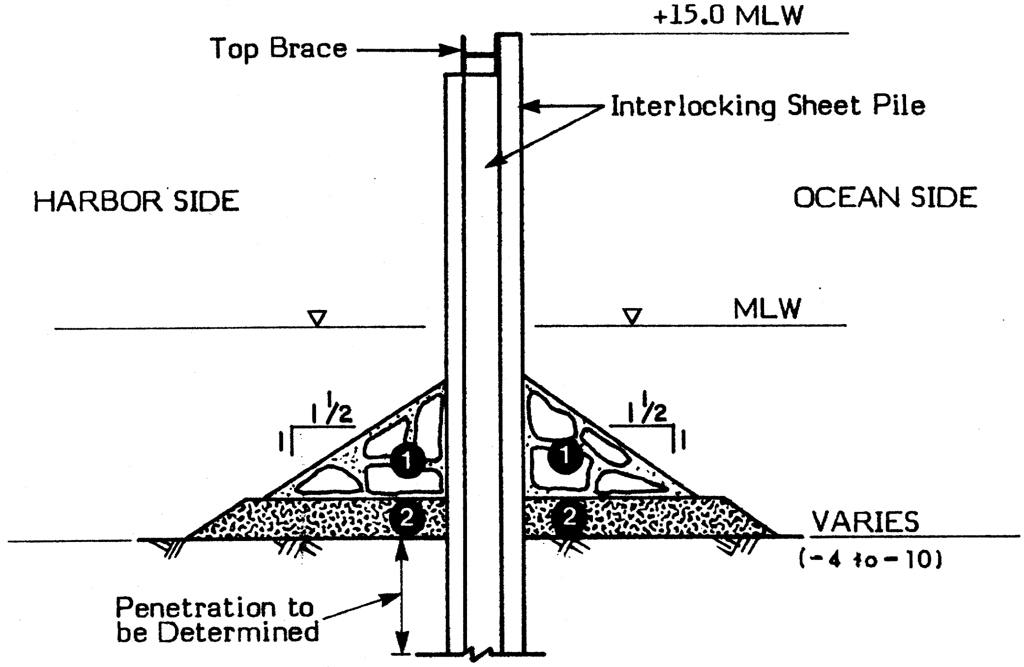
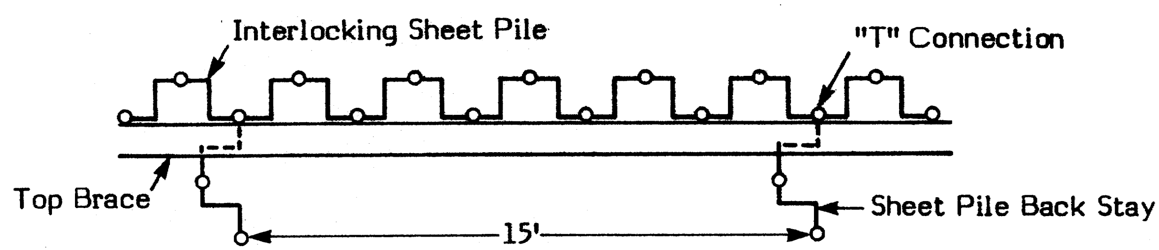
CELLULAR SHEET PILE BREAKWATER
EXHIBIT V-2



TYPICAL CROSS-SECTION WITH CREST ELEVATION 15' MLW Scale 1"=10'



PLAN Scale 1"=5'

MATERIALS: Interlocking Sheet Pile
I-Beam or Channel Top Brace
❶ 2 Ton Armor Stone,
❷ Bedding Stone, 2.5"-8"

CREST ELEVATION: Minimum 15' MLW will meet design wave criteria

THIN WALL SHEET PILE BREAKWATER

EXHIBIT V-3

The flexible concrete wall is composed of the following:

a) Steel piles in the shape of an A-frame connected with a structural tubing running in the direction of the longitudinal axis of the breakwater.

b) A precast-prestressed concrete wall panel which would be suspended from the structural tubing by means of flexible hinges, to allow for movement in a direction normal to the longitudinal axis of the breakwater. Based on the forces calculated for a fixed type structure, the frame spacing would be approximately 9 feet and the corresponding concrete panel width would be approximately 6 feet. The length of concrete panel, depending on location along the breakwater, would vary from 19 to 23 feet. The flexible concrete wall was eliminated from consideration because of structural cost feasibility and inability of the structure to provide the necessary wave attenuation.

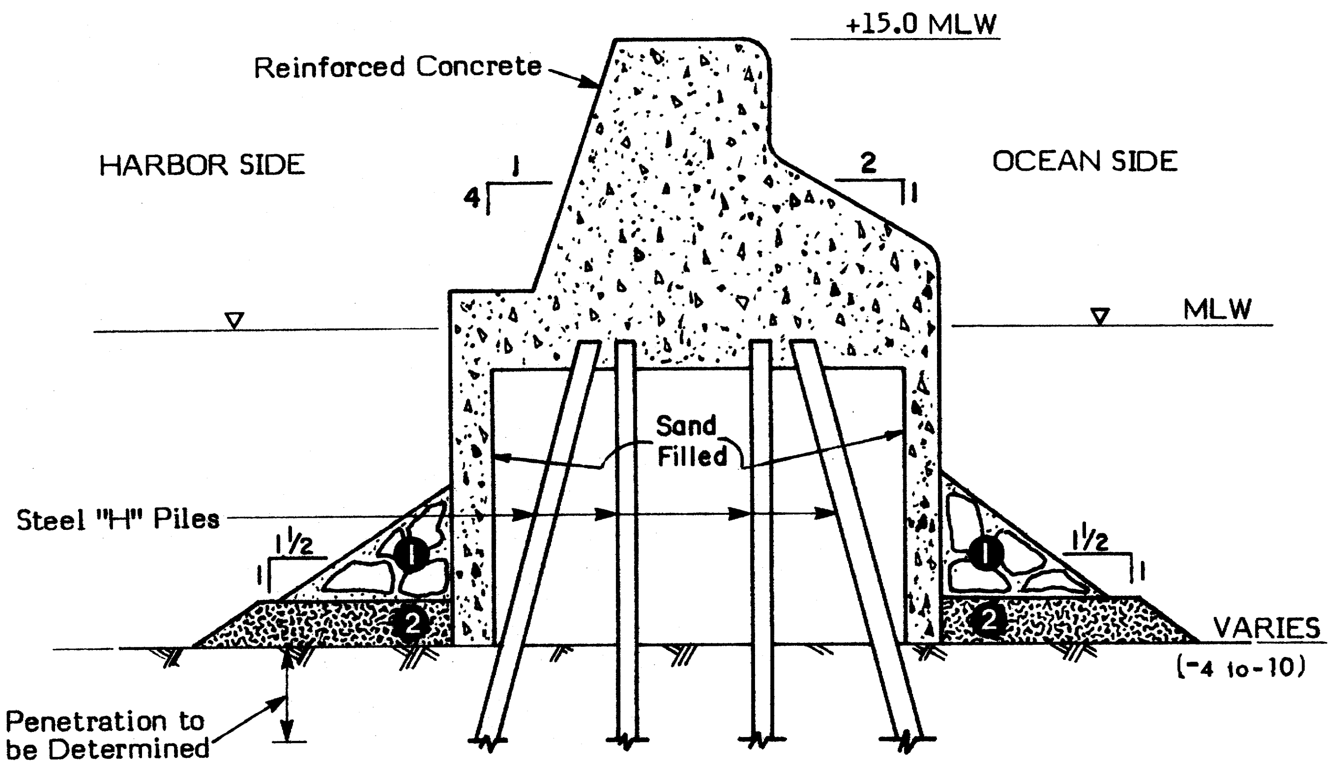
Exhibit V-4 shows a schematic design of a concrete composite slope breakwater that is feasible for the protection of Black Rock Harbor. The shape is designed with a hollow core to limit concrete requirements and to reduce foundation loads. Support is provided by deep driven steel H-piles beneath the structure. The composite slope of the face is shaped to provide for rapid breaking waves and to limit overtopping. Again, 2-ton stone or armor units are to be used for toe protection and a crest elevation of at least 15 ft is recommended from stability considerations.

Floating Breakwaters

Recent advances in the design of floating breakwaters have led to their application in marine environments. For this reason and because of potential economies of such an approach, two such breakwaters were considered for Black Rock Harbor. These were a pontoon breakwater and a floating tire breakwater.

Exhibit V-5 shows a preliminary dimension sketch of a pontoon floating breakwater for use in the Harbor. The exact dimensions of the units and construction details would require detailed study if they were selected. In this instance, two forms are considered. The first would be of reinforced concrete construction. An alternative would be welded steel construction. In either case, the vertical and horizontal dimensions would be similar and the pontoons would be foam filled to insure flotation.

Exhibit V-6 presents a floating breakwater constructed of used tires. This design features a submerged leading edge to induce wave breaking, curtains beneath the body of the breakwater to damp oscillatory motions, and a scalloped trailing edge to limit wave formation on the back side of the breakwater.



TYPICAL CROSS-SECTION WITH CREST ELEVATION 15' MLW Scale 1"=10'

MATERIALS: Reinforced Concrete
Steel "H" Piles
❶ 2 Ton Armor Stone,
❷ Bedding Stone, 2.5"-8"

CREST ELEVATION: Minimum 15' MLW will meet design wave criteria

Alternative Breakwater Study
BLACK ROCK HARBOR ENTRANCE
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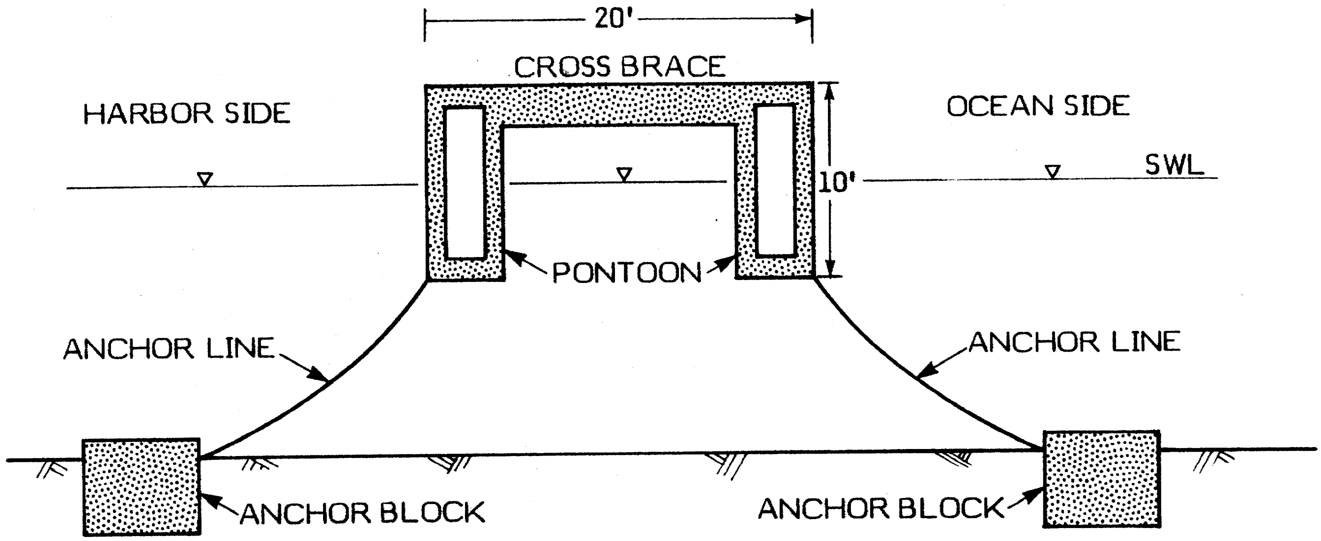
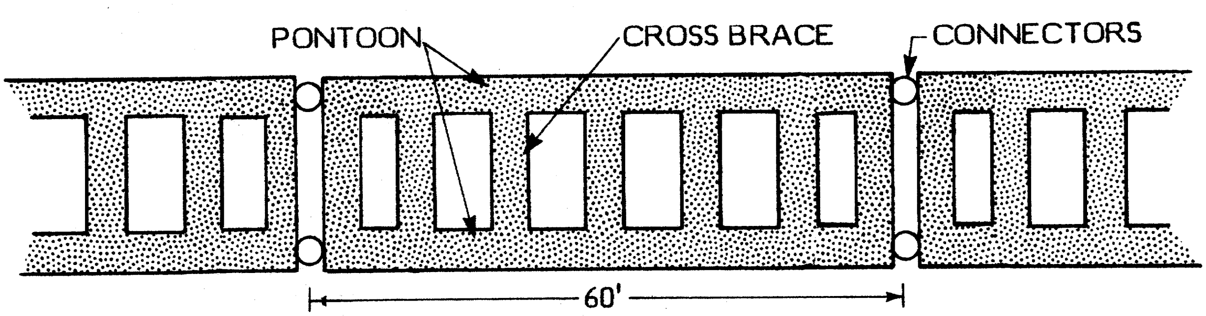
CONCRETE COMPOSITE SLOPE BREAKWATER
EXHIBIT V-4



TYPICAL CROSS SECTION

Scale 1" = 10'



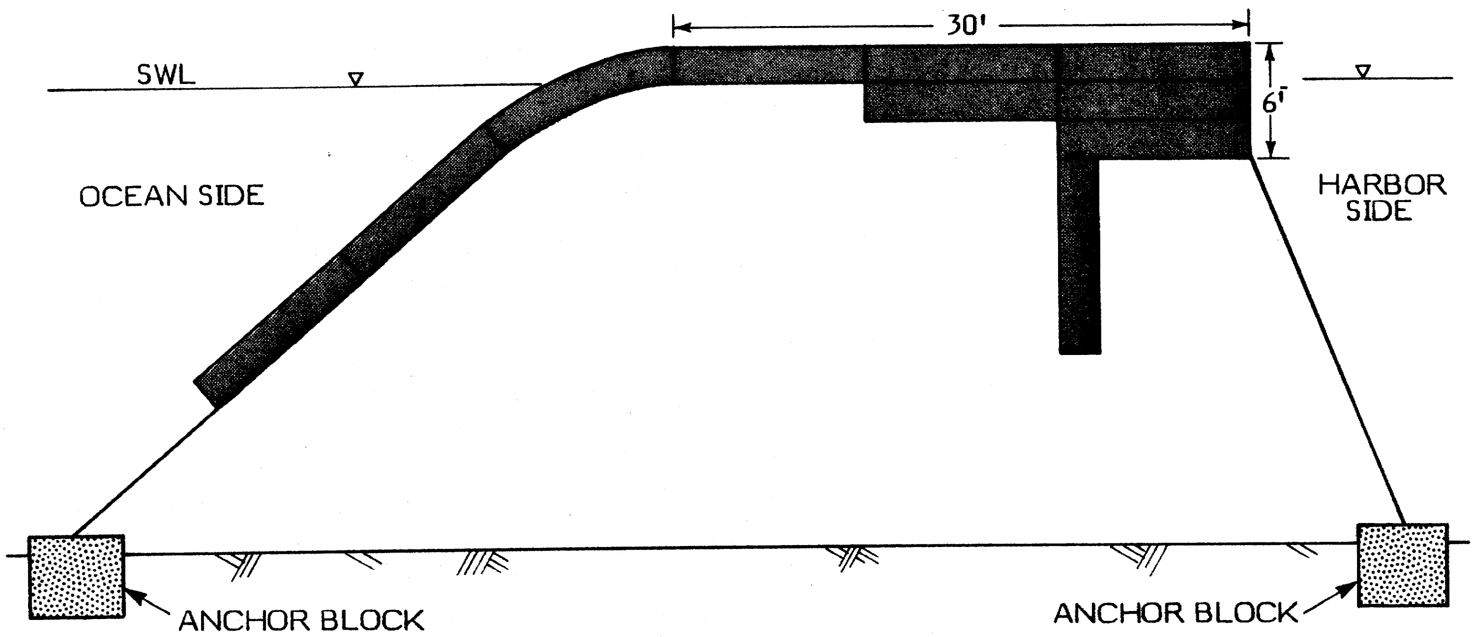
PLAN

Scale 1" = 20'

MATERIALS: Pontoon sections and cross bracing can be of pre-cast concrete or of welded steel. Anchor and connector must be specifically designed, as must detail of cross bracing.

CATAMARAN PONTOON BREAKWATER

EXHIBIT V- 5



CROSS SECTION

Scale 1" = 10'

MATERIALS: The basic "Building Blocks" shown above are tire mats of two tire thickness and four tires width. Length can vary, but 20 tires would be simple to handle and place.

FLOATING TIRE BREAKWATER

EXHIBIT V-6

As promising as these designs appeared, both were rejected for further consideration for a variety of reasons. Foremost among these is the inability of floating breakwaters to provide the needed wave reduction (approximately 80 percent) required to protect Black Rock Harbor from the design storm wave. Secondary considerations included the large mooring forces, discussed in detail in Section IX, encountered during the design storm, durability problems encountered with such structures in similar environments, and reluctance of suppliers to recommend floating structures for 6 ft design waves.

Recommended Breakwater Types

Of the several types of breakwaters considered for Black Rock Harbor, only four showed sufficient promise to be carried forward into more detailed study. The traditional rubble mound structure, with either stone or armor unit protection, could provide suitable wave reductions. Either the cellular sheet pile structure or the thinwalled sheet pile structure could provide similar protection and could offer cost advantages in the poor foundation conditions encountered at the Harbor. Finally, the concrete composite slope breakwater was carried forward because it can be entirely pile supported and offers substantial wave reduction capabilities.

SECTION VI

INCIDENT WAVES

The preceding sections have described the design storm for Black Rock Harbor, assessed a range of possible locations for the breakwaters, and compared various types of breakwater structures. This has led to the selection of three possible locations (W1, E1, and E2) and four recommended types of structures for detailed consideration. The purpose of this section is to determine the characteristics of the waves that will impinge upon the breakwater locations during the design storm and to determine the forces that each type of breakwater must resist.

The first step in this process is accomplished by examining the refraction diagrams previously presented. From the refraction coefficients for the breakwater locations, the height, period, and wavelength for the critical design wave for each location is calculated. Then the techniques in the Shore Protection Manual are used to determine the forces on the various breakwater types generated by the design waves.

Refraction Coefficient, Incident Wave Height

The incident design wave parameters for each breakwater are provided in Exhibit VI-1. The critical wave for the west breakwater is generated by winds from the south-southeast. The average refraction coefficient along the breakwater for this wave is 0.73, providing an incident significant wave height of 5.1 ft which is rounded up to 5.5 ft. The design wave for both of the east breakwater alternatives is generated by southeast winds. The average refraction coefficient for the E1 alternative is 0.53, giving a significant wave height of 4.24 ft. which is rounded up to 4.5 ft. The average refraction coefficient along the E2 alternative is 0.64, giving a significant height for the incident wave of 5.12 ft. which is rounded up to 5.5 ft. The shallow water wave lengths were computed using the deepwater wave lengths and average water depths along the breakwater.

EXHIBIT VI-1

DESIGN WAVE PARAMETERS

Breakwater	Deepwater Wave	Incident Wave
West 1	Direction: SSE H_o : 7.0 ft T : 5.6 sec L_o : 161 ft	Angle: 45° H_s : 5.5 ft H_{10} : 6.5 ft L : 123 ft
East 1	Direction: SE H_o : 8.0 ft T : 6.0 sec L_o : 184 ft	Angle: 90° H_s : 4.5 ft H_{10} : 5.5 ft L : 135 ft
East 2	Direction: SE H_o : 8.0 ft T : 6.0 sec L_o : 184 ft	Angle: 90° H_s : 5.5 ft H_{10} : 6.5 ft L : 135 ft

H_o = Deepwater Wave Height
H_s = Significant Wave Height
H_{10} = Average of Highest 10 Percent Waves
T = Wave Period
L_o = Deepwater Wave Length
L = Shallow Water (incident) Wave Length
Angle = Angle of Incident Wave Approach to Breakwater

SECTION VII

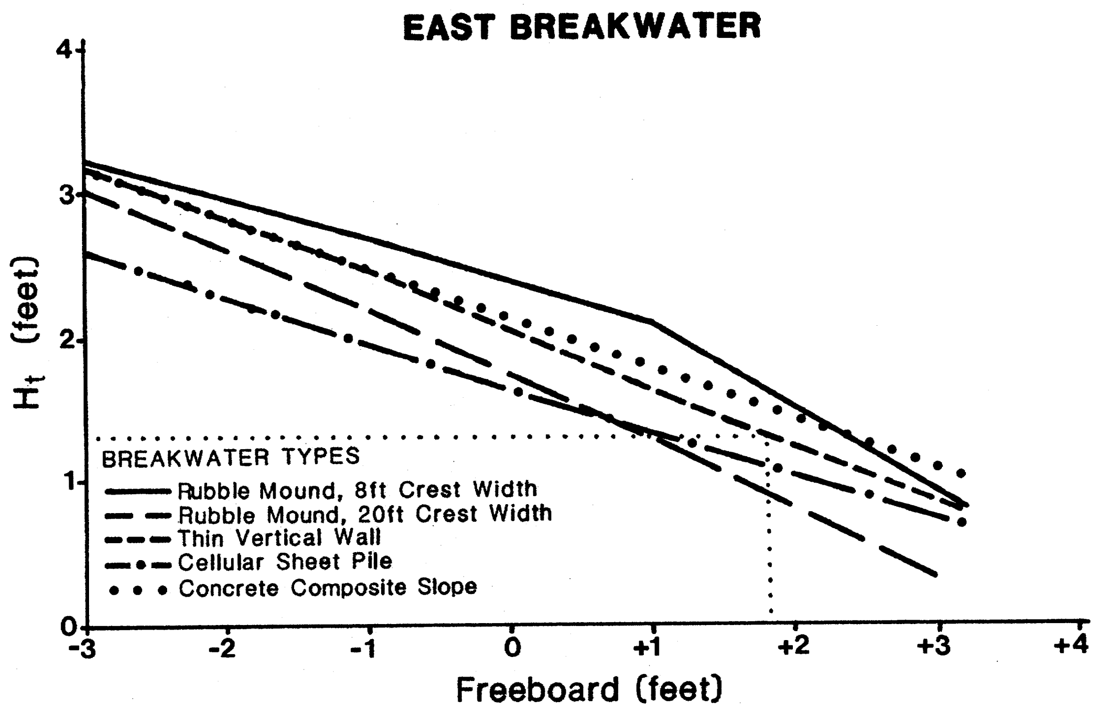
WAVE TRANSMISSION

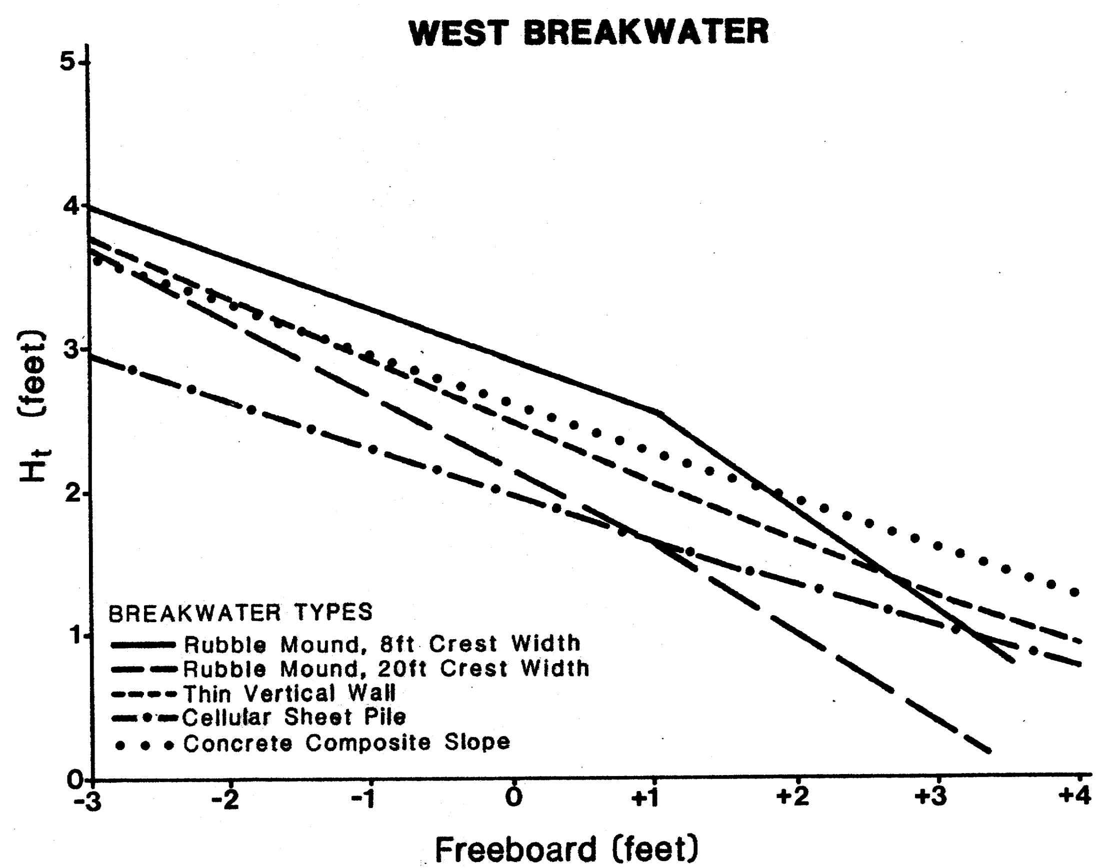
Wave transmission behind the breakwaters is primarily a function of wave runup and overtopping. All of the recommended structures allow wave overtopping to some extent. Inasmuch as the intent of the breakwaters is to provide wave protection within the Harbor, the dimensions of the structures have been based on considerations of allowable transmitted wave heights, rather than other factors commonly associated with wave runup or overtopping, such as potential flooding or damage from wave breaking at the breakwaters.

Transmitted wave heights (H_t) for the various fixed breakwater alternatives were calculated using standard methods presented in the Shore Protection Manual. Values of H_t are plotted in Exhibit VII-1 as a function of breakwater freeboard to provide an estimate of the relative advantage of increasing crest elevation for the breakwaters. The design wave values used in these calculations are presented in Exhibit VI-1. An average water depth of 20 ft MLW along the breakwaters was used (bottom elevation of -7.0 ft MLW).

H_t values were calculated for two widths of rubble mound breakwater: one with an 8 ft crest width and another with the crest width equal to the average water depth (20 ft). H_t values for the rigid, thin vertical wall sheet pile breakwater were considered to approximate those for the flexible concrete wall sheet pile breakwater. Although wave transmission through overtopping may potentially be decreased with the flexible structure, this difference is probably relatively small and would be partially compensated by energy transmission through the flexible structure.

Use of Exhibit VII-1 for estimating the necessary breakwater size, given the breakwater type and allowable H_t, is presented in the following example. If a transmitted wave height of 1.3 ft is determined to provide design conditions behind the east breakwater, the necessary height of the breakwater may be found by entering the upper figure in Exhibit VII-1 at H_t = 1.3 ft and reading the necessary freeboard from the curve for the given breakwater type. For example, a thin vertical wall breakwater would provide the necessary H_t with a freeboard of approximately 1.8 ft, which may be rounded to 2 ft (15 ft above MLW).





Alternative Breakwater Study
BLACK ROCK HARBOR ENTRANCE
Bridgeport, Connecticut

BREAKWATER FREEBOARD vs. TRANSMITTED WAVE HEIGHT

EXHIBIT VII-1

It has been assumed that design surge water levels will prevail behind the breakwaters, particularly given the substantial gaps designed for the ends of the breakwaters as well as the boating channel. The advantages of increased tidal exchange and superior harbor flushing offered by this design outweigh any marginal decrease in design surge flood levels that may be achieved with shore connected breakwaters.
All breakwaters have been designed to withstand the wave forces associated with runup and overtopping under design storm conditions. In the case of the rubble mound and cellular sheet pile alternatives, this criterion is satisfied by the placement of armor stone on the Harbor side and/or top of the structure.

SECTION VIII

DESIGN STORM WAVE HEIGHT DISTRIBUTION IN HARBOR

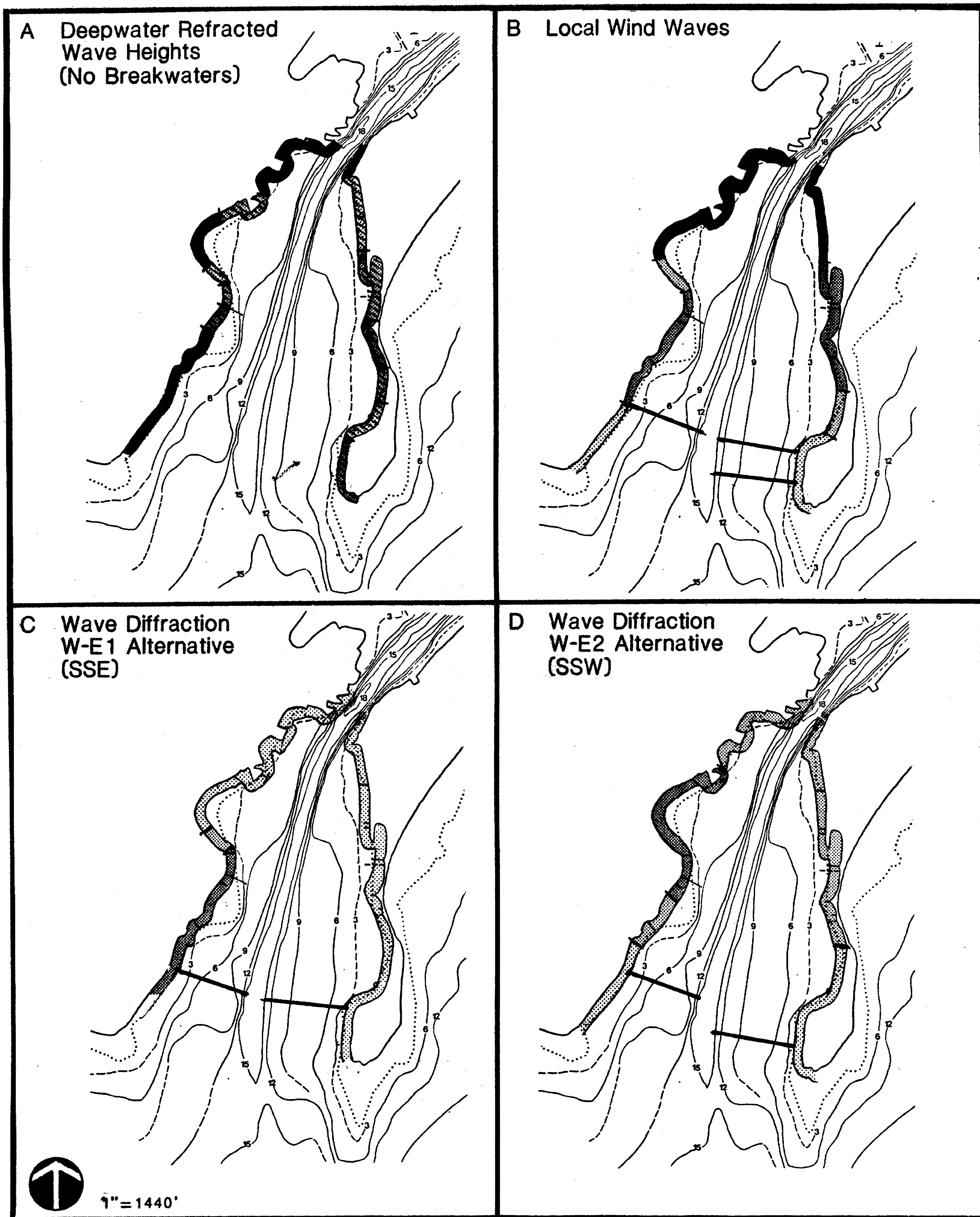
An evaluation of the feasibility of breakwater construction at Black Rock Harbor required information on the wave heights within the harbor under both existing conditions and with breakwaters. The critical wave conditions in the harbor under the existing conditions are produced by deepwater waves arriving from Long Island Sound. Critical wave conditions after breakwater construction can potentially develop from local wind wave generation within the harbor, from diffraction of deepwater waves through the breakwater gap or from waves transmitted over the breakwater under design storm conditions. The breaking wave height distribution around the harbor has been calculated for all of these wave types and for all directions of wind and wave approach considered in the design storm formulation. For the purposes of comparing the wave conditions and identifying the source of critical wave heights in the harbor, a composite of the maximum wave heights around the harbor for all directions of wind or wave approach was compiled for refracted and diffracted deepwater waves and for local wind waves. Maps of the composite maximum wave heights for the various wave types are presented in Exhibit VIII-1 and the various components of the Exhibit are discussed below.

Existing Conditions

Exhibit VIII-1 presents a composite of maximum breaking wave heights around Black Rock Harbor relevant to the selection of breakwater alternatives. Exhibit VIII-1A presents the composite maximum breaking wave heights in the Harbor with no breakwaters. As discussed in the Design Storm section, maximum wave heights from design storm deepwater waves are largely concentrated along the outer Harbor shore (Exhibits III-4, III-5, and III-7). Waves generated by south-southeast winds produce the largest waves along the western side of the Harbor while waves generated by south-southwest winds produce the largest waves along the eastern side of the Harbor. Wave heights from refracted deepwater waves are considerably smaller at the head of the Harbor.

Waves Generated Within the Harbor

Exhibit VIII-1B presents a composite of the maximum wave heights that may be generated behind the breakwaters by winds of 42 knots from the southeast to southwest. These waves were estimated using an average water depth of 20 ft (13 ft storm surge) and forecasting relationships given in the Shore



Alternative Breakwater Study
BLACK ROCK HARBOR ENTRANCE
Bridgeport, Connecticut

COMPOSITE MAXIMUM WAVE HEIGHTS AT BREAKING

0'-1'	2'-2.5'	4'-5'
1'-1.5'	2.5'-3'	5'-6'
1.5'-2'	3'-4'	>6'

EXHIBIT VIII-1

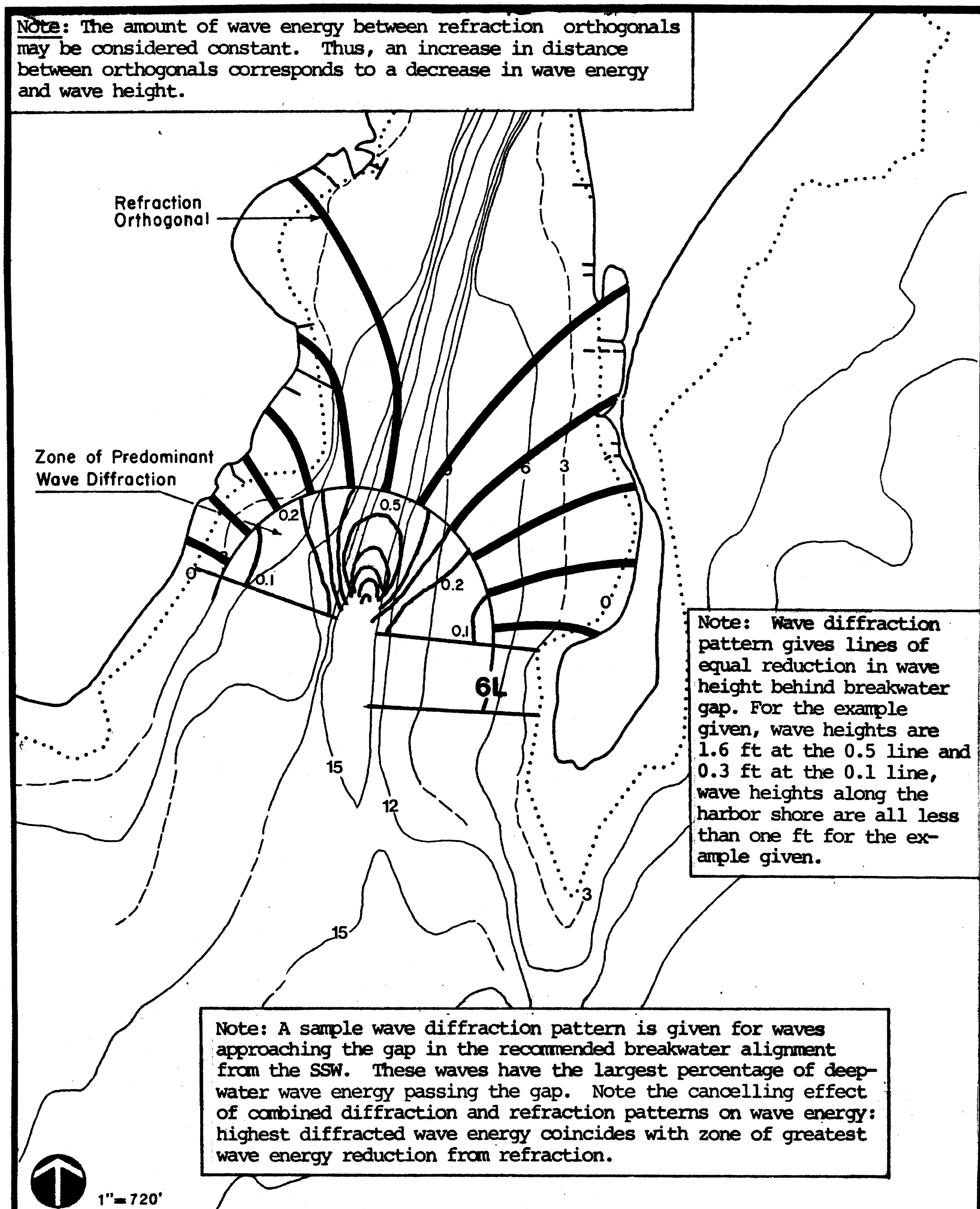
Protection Manual. It may be seen that the maximum local wind waves are considerably smaller than the deepwater waves under existing conditions, but are still of significant size. Local wind waves up to 2.5 ft in height are found at the head of the Harbor, reflecting the longer fetch in this direction. Wave heights greater than 1.5 ft are also found along the outer shore of either side of the Harbor. These local wind waves may be generated behind any breakwater alternative and consequently play a role in defining the necessary size of the structures. A breakwater which decreases the incident deepwater wave heights below this level would not significantly improve wave conditions within the Harbor and would thus be overdesigned.

Diffraction by Breakwaters

Maximum breaking wave heights associated with diffraction of deepwater waves through the breakwater gaps are presented in Exhibit VIII-1C for the W1-E1 alternative and in Exhibit VIII-1D for the W1-E2 alternative. Diffraction of the deepwater waves was performed using standard graphical techniques. Under conditions of wave approach over a shallow bottom with contours at an angle to the direction of wave approach, it is known that the effects of wave diffraction become subordinate to those of wave refraction within several wavelengths of the breakwater gap. Consequently, a combined diffraction/refraction analysis of the waves passing through the breakwater gaps was performed. Wave diffraction was carried out to a distance of 6 wavelengths from the breakwater gap, at which point refraction of the waves was calculated into the harbor. Ten different diffraction patterns within the 6 wavelength zone behind the breakwaters were calculated for the different breakwater gaps and the 5 directions of wave approach considered. The refraction pattern behind the breakwater gap diffraction zone is presented in Exhibit VIII-2 and shows a general dissipation of the diffracted wave energy towards the head of the harbor.

The composite maximum wave height distribution for diffraction of waves through the W1-E1 gap is identical to that for only waves generated by winds from the south-southeast, the critical storm wave for that breakwater alternative. It may be noted that the breaking wave heights associated with diffraction of this wave are equivalent or less than those possible under local wind generation and are thus not considered critical.

The composite maximum breaking wave heights from diffraction through the W1-E2 gap are the same as those for waves generated only by south-southwest winds. It may be seen that wave heights around the Harbor in Exhibit VIII-1D are also less



Note: The amount of wave energy between refraction orthogonals may be considered constant. Thus, an increase in distance between orthogonals corresponds to a decrease in wave energy and wave height.

Note: Wave diffraction pattern gives lines of equal reduction in wave height behind breakwater gap. For the example given, wave heights are 1.6 ft at the 0.5 line and 0.3 ft at the 0.1 line, wave heights along the harbor shore are all less than one ft for the example given.

Note: A sample wave diffraction pattern is given for waves approaching the gap in the recommended breakwater alignment from the SSW. These waves have the largest percentage of deep-water wave energy passing the gap. Note the cancelling effect of combined diffraction and refraction patterns on wave energy: highest diffracted wave energy coincides with zone of greatest wave energy reduction from refraction.

Alternative Breakwater Study
BLACK ROCK HARBOR ENTRANCE
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REFRACTION OF HARBOR WAVES PASSING BREAKWATER GAP (Diffraction to 6L)

EXHIBIT VIII-2

than or equal to those generated by local winds except for a small section along the western shore where they are slightly higher. Inasmuch as selection of the W1-E2 alternative would be based largely on the potential for improved wave protection, it is evident in Exhibit VIII-1 that this alternative not only does not provide superior protection to the W1-E1 alternative, but also that the difference is of little relevance as local winds produce more critical wave conditions than allowed by diffraction with either alternative. As a consequence, construction of a breakwater in the more energetic wave environment at E2 cannot be justified.

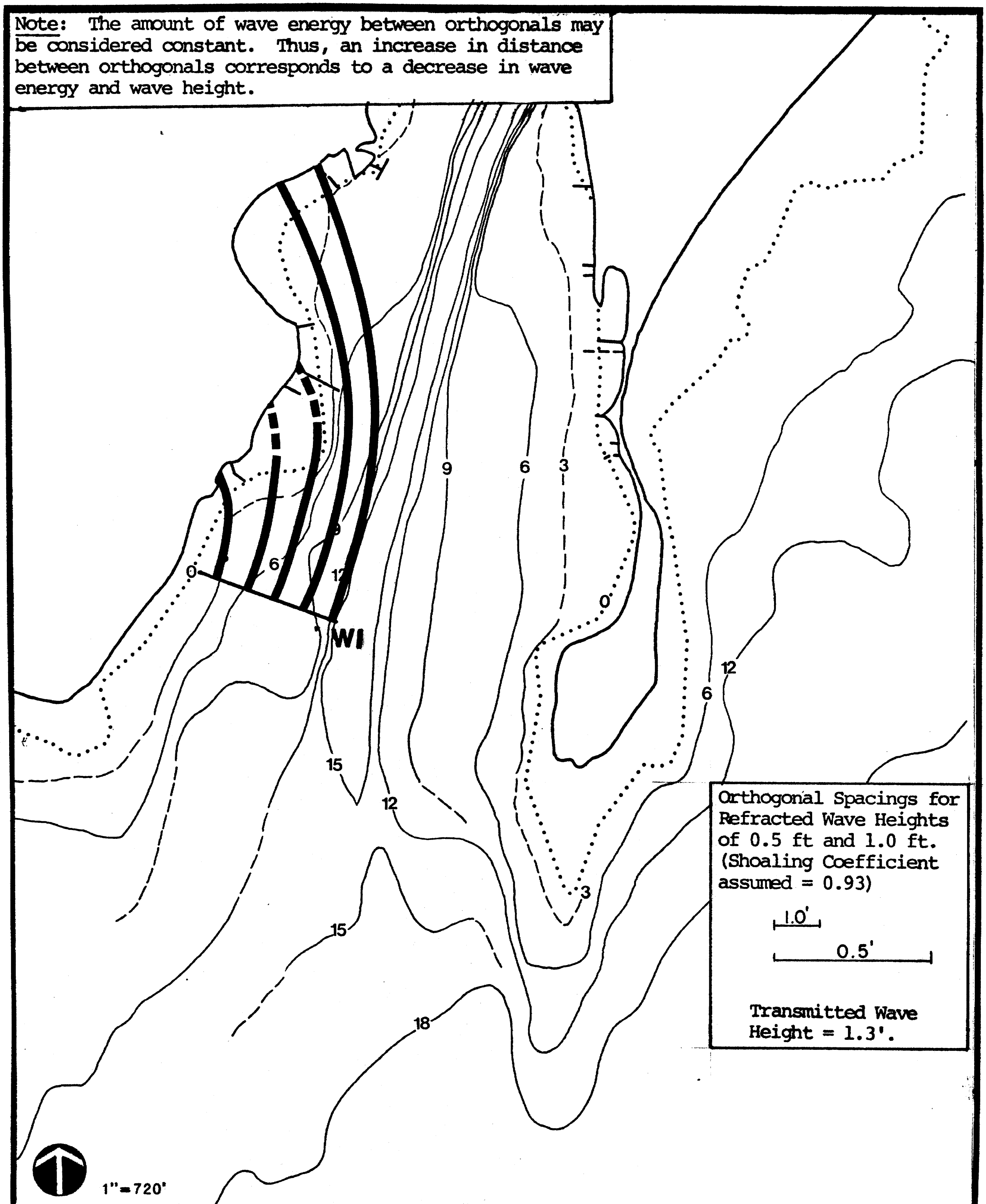
Transmitted Waves

With a preselected location of breakwater alternatives, wave conditions within the harbor related to wave diffraction and local wind wave generation are relatively determinate. Thus, it is not difficult to consider variable breakwater dimensions and related transmitted wave heights to determine the necessary breakwater size to meet design criteria. To this end, refraction analyses for transmitted waves behind each breakwater alternative were performed to evaluate their potential distribution around the harbor. The refraction patterns were calculated for the design wave parameters presented in Exhibit VI-1, a 3 ft contour interval and a 180 ft orthogonal spacing. The results are presented in Exhibits VIII-3, VIII-4 and VIII-5.

It may be seen in Exhibits VIII-4 and VIII-5 that little dissipation of wave energy through refraction occurs along the east side of the Harbor as the waves traverse the relatively flat bottom. Towards the head of the Harbor, the wave orthogonals bend somewhat uniformly producing a refraction coefficient close to unity along parts of the eastern Harbor shore. This situation will remain if dredging of the area for improved anchorage is performed in the future.

Design Criteria

The preceding sections have shown that diffraction of deepwater waves through the breakwater gaps is of minor consequence for the conditions considered, producing wave heights within the harbor which are generally smaller than those produced by local winds. Under these conditions, waves transmitted over or through the breakwaters become the controlling factor in breakwater dimension selection. For this study, the aim of protecting the Harbor will best be served if the transmitted waves are made equal to or smaller than the local waves generated within the Harbor. Any more stringent criterion would not further protect the Harbor, since the local waves would still form during a storm.

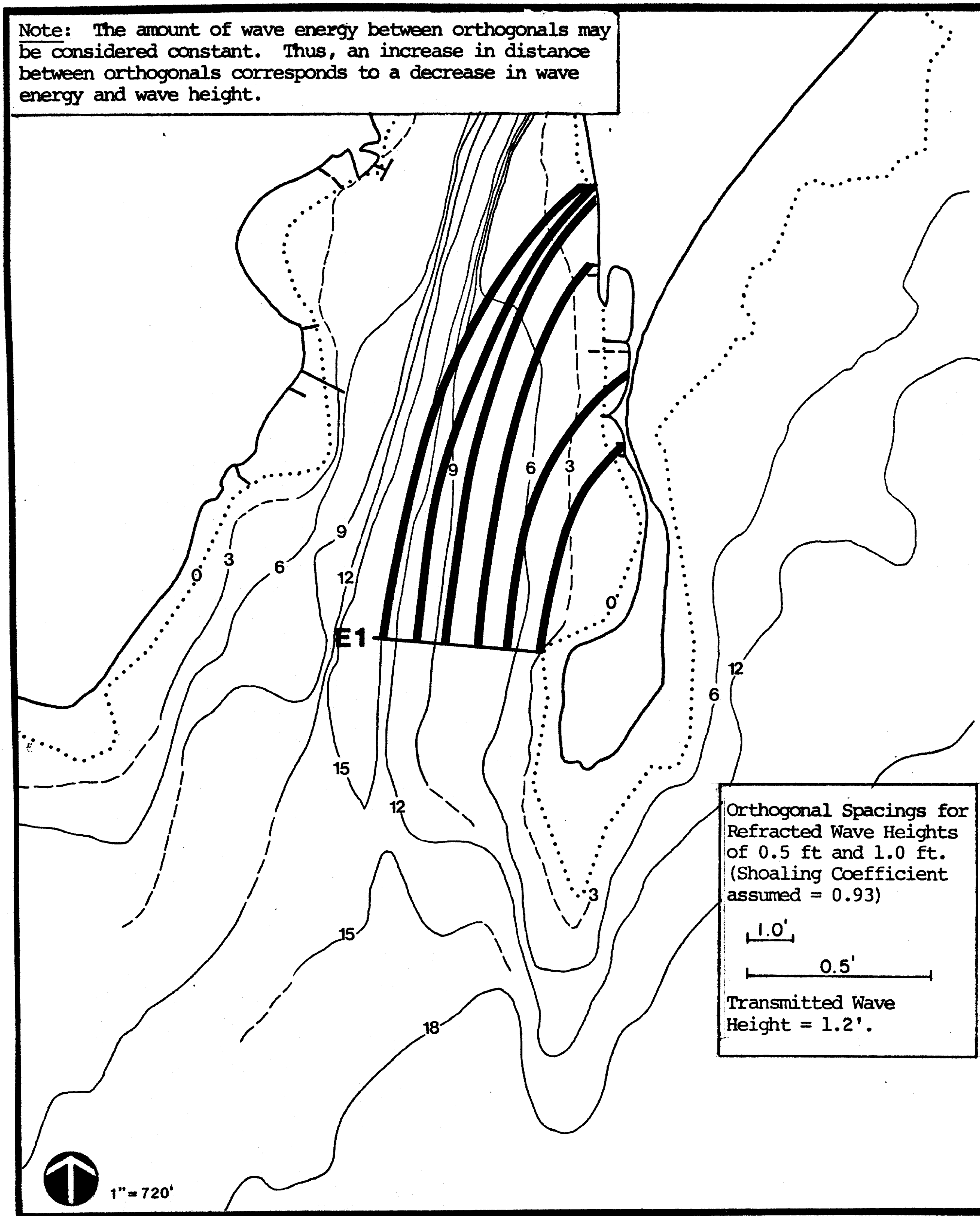


Alternative Breakwater Study
BLACK ROCK HARBOR ENTRANCE
Bridgeport, Connecticut

REFRACTION PATTERN FOR TRANSMITTED WAVES BEHIND RECOMMENDED WEST BREAKWATER

EXHIBIT VIII-3

Note: The amount of wave energy between orthogonals may be considered constant. Thus, an increase in distance between orthogonals corresponds to a decrease in wave energy and wave height.



Orthogonal Spacings for Refracted Wave Heights of 0.5 ft and 1.0 ft. (Shoaling Coefficient assumed = 0.93)

1.0'

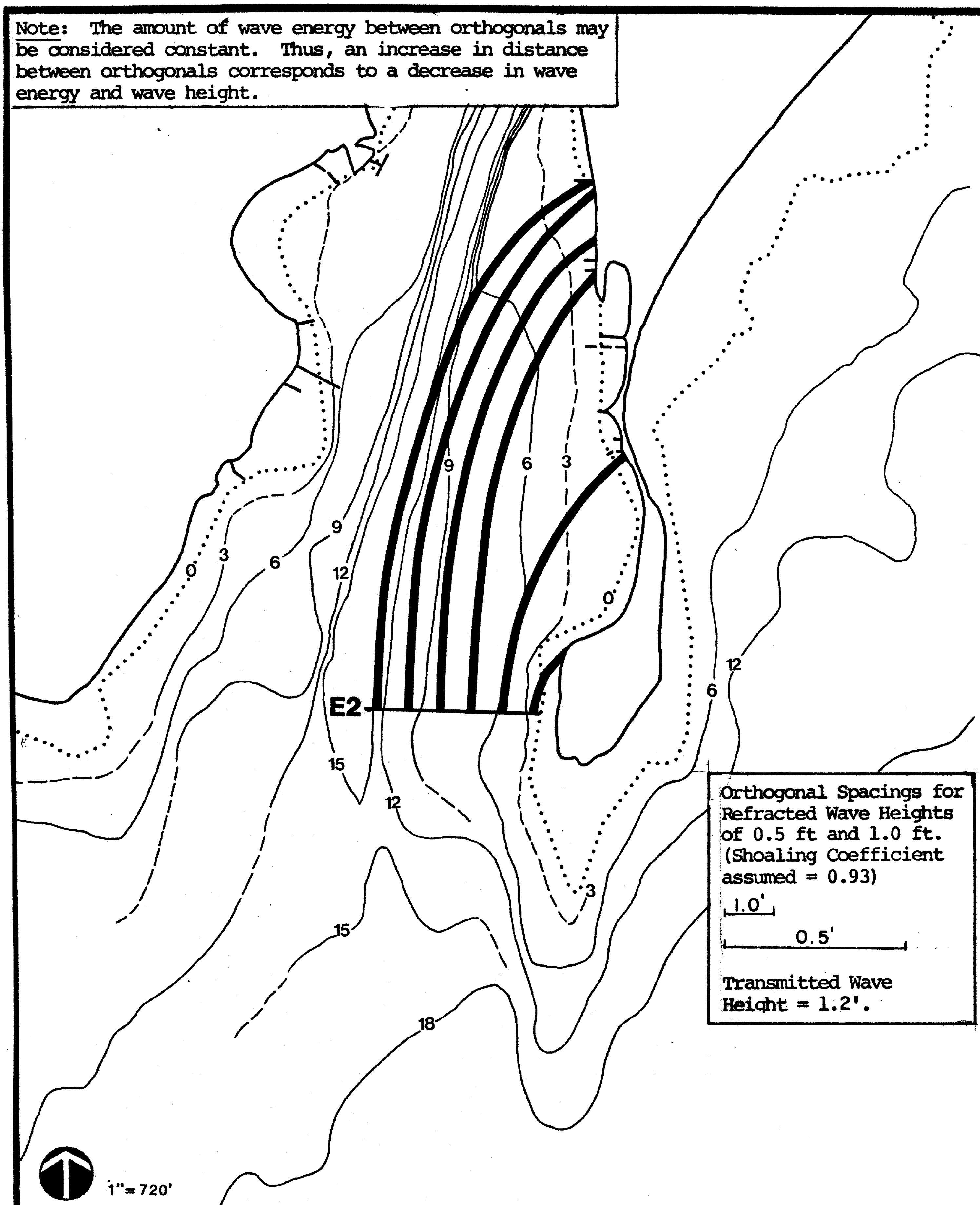
0.5'

Transmitted Wave Height = 1.2'.

Alternative Breakwater Study
BLACK ROCK HARBOR ENTRANCE
Bridgeport, Connecticut

REFRACTION PATTERN FOR TRANSMITTED WAVES BEHIND RECOMMENDED EAST BREAKWATER

EXHIBIT VIII-4



Alternative Breakwater Study
BLACK ROCK HARBOR ENTRANCE
Bridgeport, Connecticut

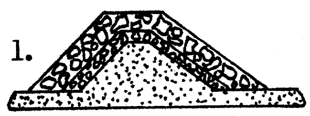
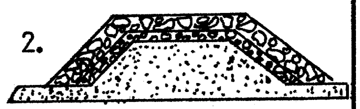
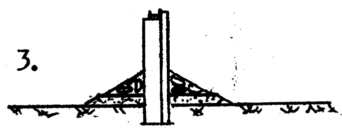
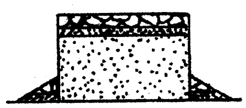
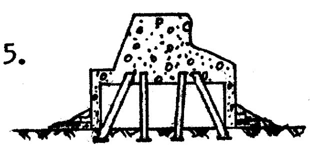
REFRACTION PATTERN FOR TRANSMITTED WAVES BEHIND ALTERNATIVE EAST BREAKWATER

EXHIBIT VIII-5

The height of the allowable transmitted wave was determined using the refraction patterns given in Exhibits VIII-3 to VIII-5 and the local wind wave height distribution in Exhibit VIII-1B. It was found that a transmitted wave height of 1.2 ft behind the west breakwater reduced breaking wave heights along the west side of the Harbor to a level equivalent to or less than those generated by local winds. Similarly, a transmitted wave height of 1.3 ft behind either of the east breakwaters reduced breaking wave heights to the design level. This criteria, used with the wave transmission diagrams in Exhibit VII-1, allowed the direct selection of design heights for each of the four types of breakwater. These are summarized in Exhibit VIII-6. In each case, the freeboard taken from Exhibit VII-1 is added to an assumed storm surge of 13 ft to determine the required crest elevation. In reducing both transmitted and diffracted wave heights to a level less than or equal to that produced by local winds behind the breakwaters, the critical wave environment within the harbor with construction of the breakwaters to the dimensions given in Exhibit VIII-6 will be equivalent to that shown in Exhibit VIII-1b.

EXHIBIT VIII-6

RECOMMENDED BREAKWATER CREST ELEVATIONS

West Breakwater			East Breakwater	
Freeboard (ft)	Crest Elevation (MLW)	Breakwater Type	Freeboard (ft)	Crest Elevation (MLW)
2.9	16.0	1. 	2.4	15.5
1.7	15.0	2. 	1.0	14.0
3.1	16.0	3. 	1.8	15.0
2.4	15.5	4. 	1.1	14.0
4.1	17.0	5. 	2.4	15.5

Breakwater freeboard designed for a transmitted wave height of 1.2 ft for west breakwater and 1.3 ft for east breakwater.

Breakwater Types:

1. Rubble Mound, 8 ft crest width
2. Rubble Mound, 20 ft crest width
3 Thin Vertical Wall
4. Cellular Sheet Pile
5. Concrete Composite Slope

SECTION IX

STRUCTURAL CONSIDERATIONS

An analysis of the structural and foundation requirements of the five alternative breakwater types shown in Exhibit VIII-6 was made to establish their critical elements and to provide information for a construction cost estimate.

Forces on the rubble mound breakwaters are not of great concern in design, except for the selection of the armor stone. Static loading of the foundation materials will be an important factor in consideration of differential settlement. The design loads to be carried by a thin vertical wall, cellular sheet pile or composite slope breakwater were computed from the static and dynamic pressures created by the striking waves at a storm surge water elevation of thirteen feet (+13) above mean low water. The computation used a design wave height of 5.5' with an angle of incidence (α) of 45^{o} for the west breakwater and a height of 5.5' with an α of 90^{o} for the east breakwater. For the computation of the resultant forces and overturning moments, the composite slope breakwater is assumed to behave as a vertical wall under wave action. The variations in load distribution resulting from a breakwater shape different from a vertical wall were discounted since a model analysis is required to determine the precise distribution. A vertical wall assumption was judged to be sufficient for the structural and foundation requirement analysis at this level of design study.

The computed loads are:

- West Breakwater F = 5.0 Kips/lin. ft.
  M = 58.0 foot-kip/lin. ft.

- East Breakwater F = 7.0 Kips/lin. ft.
  M = 79.0 foot-kips/lin. ft.

The foundation requirement analysis was based on the available soils information, which consisted of a group of manual probings and a line of borings taken in one location in the breakwater study area in 1954* (see Exhibits IV-1 and IV-2). The explorations indicate a layer of organic silt of increasing thickness from the shore to the channel. At the channel, the silt extends to about elevation -50. The foundation requirement analysis was, therefore, based on the assumption that firm soil strata capable of carrying breakwater loads exist at an average elevation of -40.

* Bridgeport Harbor-Black Rock Harbor Soil Boring and Probings Plan and Geologic Section, Corps of Engineers, May 24, 1954.

Rubble Mound (Exhibit V-1)

Breakwaters of this type are sensitive to large differential settlelment and, in Black Rock Harbor, would require the removal of all or some of the organic silt and replacement with granular fill to provide a uniform foundation for the rubble mound. The configuration, thicknesses and unit weights of the armor layers was selected for the design wave and overtopping forces.

Cellular Sheet Pile Breakwater (Exhibit V-2)

The cell diameter required to prevent overturning or sliding and to limit center of cell shear values and sheet interlock tension values to allowable limits was determined to be 25 feet. The cell construction would not require removal of the organic silt, but the sheet pilings would have to be driven through the silt into firm underlying strata or rock.

Thin Wall Sheet Pile Breakwater (Exhibit V-3)

The sheet piles of this type of breakwater would also have to be driven through the organic silt into firm underlying strata or rock. Bracing design would vary with the depth of organic silt. Battered pile braces could be used near the shore where the organic silt layer is shallow and where bedrock is close to the harbor bottom (west side). Straight pile braces, as shown on Figure V-3, would be required in the deep organic silt adjacent to the channel as battered pile braces would be excessively long and straight pile braces would be needed to stiffen, as well as support, the 50' to 60' height of sheet pile breakwater.

Concrete Composite Slope Breakwater (Exhibit V-4)

The breakwater loads would be transmitted directly to firm underlying soils or rock by piles with an assumed capacity of 70 tons. Each row of four piles would be set about 10' apart. Excavation of a portion of the underlying organic silt and replacement with granular fill would be required to support the precast concrete box sections surrounding the piles during construction.

Floating Breakwater - Catamaran Pontoon/Tire (Exhibits V-5, V-6)

Exhibit VI-1 presents a design wave height of approximately 6 ft. and a wave length of approximately 130 ft. Current manufacturers of flotation devices recommend that floating breakwaters be restricted to applications where wave heights are 3 to 4 ft. maximum. In addition, the width of the floating device should approximate 0.6 of the design wave length. In Black Rock Harbor, this would produce a desired beam width of 80 ft.

The floating breakwater shown in Exhibit V-5 provides a beam width of 20 ft., the maximum size presently manufactured. Analysis of this section shows that approximately 80% of the structure would be below water. At mean low water, a major portion of the breakwater would be resting on the bottom of the harbor and at low tide, the entire breakwater would be resting on the harbor floor. This condition would create a stress distribution on the structure not accounted for in the structural design.

The mooring system includes a cable system and concrete block anchor. Based upon the determined wave forces, the cable forces will approximate 15,000 lbs. requiring a one inch diameter cable and a concrete block of approximately 15 tons in size. A mooring block of this size resting on the harbor floor will penetrate and sink into the soft silty soil until it reaches firm substrata. Mooring cable lengths would be effected by this settlement.

SECTION X

CONSTRUCTION CONSIDERATIONS AND COSTS

The accessibility of the site for construction equipment and material delivery, and the availability of necessary construction material, are the primary construction considerations affecting the selection of the type or types of breakwater to be installed.

Access to the proposed breakwater site on the west shore is constrained by the residential development of the area, the irregular pattern of streets and the high steep slope along the shore. The use of the residential streets by large trucks delivering materials would probably be unacceptable to the community. Access to the breakwater site on the east side is constrained by the passable, but narrow sand spit connecting Fayerweather Island to the mainland. The access constraints, and/or the design of each alternative type of breakwater under study, will require the use of marine equipment for construction. Construction with land equipment, and the construction of temporary fills or structures to facilitate the use of land equipment, may be necessary close to the shore where shallow water would prevent the use of marine equipment without dredging.

Rubble Mounds

The material for the core and armor subbase is available in Connecticut, but the large armor stones would have to be obtained from quarries in Rhode Island or Massachusetts. Due to the demand for aggregate stone, Connecticut quarries are reluctant to produce large blasted rock. The breakwater design requires construction to be accomplished by marine equipment except close to shore. Only the placement of the armor stone on a breakwater with a 20' wide top would be practical with land equipment and truck delivered stone.

Thin Wall Sheetpile

The placement of the sheet piling and submerged armor protection would have to be almost entirely accomplished with marine equipment. Placement by land equipment near shore could only be accomplished from a temporary platform or causeway. The longer lengths of sheet piling (over 40') required for sections near the channel underlain by deep organic silt would have to be shipped by rail and transferred to barges for delivery to the site.

Cellular Sheet Pile

Sheet pile cells could be constructed progressively from shore by land equipment, but as noted above, sheet piles over about 40' would have to be delivered by barge.

Concrete Composite Slope

As described for the thin wall sheet pile breakwater, this type would also have to be constructed almost entirely by marine equipment, including a concrete plant, due to its size and shape.

The construction cost of each alternative type of breakwater at each alternative location has been estimated from the lengths shown on Exhibit IV-2 and the structural, foundation and construction considerations previously described.

		ALTERNATIVE LOCATIONS		TOTAL CONSTRUCTION COST
TYPE		WEST 1	EAST 1	WEST 1 & EAST 1
Rubble Mound (8')	Concrete Armor	$ 4,520,000	$4,897,000	$9,417,000
	Stone Armor	2,230,000	2,415,000	4,645,000
Rubble Mound (20')	Concrete Armor	4,956,000	5,369,000	10,326,000
	Stone Armor	2,463,000	2,668,000	5,199,000
Thin Sheet Piling		2,142,000	2,320,000	4,462,000
Cellular Sheet Piling		4,161,000	4,505,000	8,666,000
Composite Structure		2,026,000	2,198,000	4,224,000

MAINTENANCE COSTS

The determination of the extent of maintenance for the individual breakwater types is a difficult one. Factors that will affect maintenance costs include the following:

A) Construction details and considerations incorporated into the breakwater design for the reduction of maintenance.

B) Choice of materials for construction.

C) Quality control of the construction process.

D) Quality of annual inspection for needed maintenance.

E) Severity and occurrence of storms.

For Black Rock Harbor, a conservative approach was used for analysis. The cost for maintinaing the various breakwater types has been estimated using a ten year time frame which will account for annual maintenance and the occurrence of the design storm once every ten years on average. For the rubble mound, annual maintenance will consist of minor replacement (approximately 2%) of the stone or concrete armor with a major rehabilitation (approximately 10%) project every ten years. The sheet pile breakwaters will require a protective coating of the cellular and thin wall sheet piling every five years and a major reconstruction (assume two cell replacement for cellular type and 100' replacment of thin wall type) every ten years. The concrete composite structure will require spall and crack repair (assume 1% of surface area/year) and a major replacement (assume 100 ft.) of the reinforced concrete section every ten years.

The maintenance cost for alternative locations W1-E1 and the various breakwater types is as follows:

RUBBLE MOUND	ANNUAL COST	MAJOR RECONSTRUCTION	TOTAL 10 YEAR COST
8 ft. crest			
concrete armor	$ 80,000	$ 400,000	$ 1,200,000
stone armor	50,000	180,000	680,000
20 ft. crest			
concrete armor	90,000	440,000	1,340,000
stone armor	60,000	220,000	800,000

CELLULAR SHEET PILING:	PROTECTIVE MAINTENANCE	MAJOR RECONSTRUCTION	TOTAL 10 YEAR COST
	$90,000	$250,000	$430,000

THIN WALL SHEET PILING:	PROTECTIVE MAINTENANCE	MAJOR RECONSTRUCTION	TOTAL 10 YEAR COST
	$90,000	$200,000	$380,000

CONCRETE COMPOSITE STRUCTURE:	ANNUAL COST	MAJOR RECONSTRUCTION	TOTAL 10 YEAR COST
	$50,000	$120,000	$620,000

SECTION X1

ENVIRONMENTAL CONSIDERATIONS

The identification of impacts associated with the construction of Breakwaters for Black Rock Harbor fall into two categories: short term and long term. Short term impacts are normally those that occur during the construction period of the project while long term impacts are a result of project construction and may be of a permanent nature.

SHORT TERM

POSITIVE IMPACT

A. Breakwater construction will increase employment opportunities in the Bridgewater area.

B. The use of local quarries, steel companies and other businesses for construction materials will increase economic development in the Southern Connecticut area.

NEGATIVE IMPACT

A. Construction dredging could potentially produce a temporary impact on the water quality of the harbor through increased turbidity, dissolved oxygen reduction and possible release of contaminants. Necessary sediment testing, design techniques and construction control procedures should be implemented to assure that water quality standards are maintained.

B. Dredging and fill operations related to construction occuring during the months of June through September could inhibit the spawning of shellfish life.

C. Dredging will remove or disturb shellfish beds and remove a portion of the nutrient base that marine life utilizes in the immediate vicinity of the breakwater.

D. Navigation of small boat recreational traffic and larger ships delivering to shoreline facilities within Black Rock Harbor will be constricted.

E. Adjacent residential areas of Grover Hill and Black Rock may be impacted due to the transporting of construction equipment and materials.

LONG TERM

POSITIVE IMPACT

A. The reduction in periodic storm damage to moored boats and shoreline facilities.

B. Increased sheltered anchorage areas for boat moorings.

C. Increased economic development for local marinas due to increased harbor usage.

D. Increased recreational benefits with breakwaters providing fishing opportunities.

E. Rubble mound breakwaters will provide a more diverse marine habitat, supplementing the silty bottom habitat with additional hard roack habitat.

F. Dredging disposal could provide nutrients and a habitat for burrowing types of marine life, including macro-invertebrates and possibly lobster.

NEGATIVE IMPACT

A. Dredged material may be contaminated with heavy metals and organics. Testing of the dredged material should be conducted. Disposal in off-shore or nearby shorefront areas may impact existing marine life and water quality in the dump site.

B. Dredging will remove established Eastern Oyster, clam and mussel beds affecting harvesting and spawning patterns in the immediate vicinity of the breakwater.

C. Placement of the breakwaters will alter tidal currents near the harbor mouth and may cause siltation behind the break-waters. Sediment scour near the breakwater gaps may also occur, although this impact could also decrease maintenance dredging requirements in the boating channel.

D. Harbor flushing may be reduced due to harbor constriction caused by breakwater placement.

E. Increased boating usage of the harbor may cause a decrease in water quality due to the exhausting of gasoline, oils, etc.

F. The site of the recommended breakwater locations will require archeological and historical research for impacts on harbor fortifications that were built as a defense for Bridgeport.

SECTION XII

CONCLUSIONS AND RECOMMENDATIONS

The selection of an optimum plan for breakwater protection at Black Rock Harbor includes two aspects - location of the breakwater and the type of breakwater structure.

The W1-E1 breakwater location shown on Exhibit IV-2 is recommended as the preferred location because it provides the shortest breakwater length and the optimum balance between Harbor protection and favorable wave environment at the breakwater. The W1-E2 alternative was examined for its potential to provide added protection from waves generated by south-southeast and southeast winds. It was found, however, that the distribution of breaking wave heights around the Harbor under south-southeast waves was not significantly improved by the W1-E2 alternative. More importantly, it was found that wave heights associated with diffraction of deepwater waves for either alternative were smaller than waves that could be generated within the Harbor by design storm winds. Thus, selection of the shortest and most protected breakwater location is advisable.

The crest elevations of the various breakwater types for optimum wave protection within the Harbor are presented in Exhibit VIII-6. These elevations were selected to provide attenuation of incident deepwater waves to the level of waves generated within the Harbor by local winds. It was found that a transmitted wave height of 1.2 ft. for the west breakwater and 1.3 ft. for the east breakwaters would satisfy the design criterion.

The concrete composite slope type breakwater is recommended as the preferred type for the following reasons:

- the construction cost will be about equal to or lower than the other alternative types

- the shape can provide equal or better wave reduction and overtopping resistant capabilities than the other alternative types

- the required materials are readily available

- the rigid nature of a concrete structure reduces the potential for damage should overtopping occur

- it eliminates the dredging required for rubble mound types.

We wish to point out, however, that the analysis of the alternative breakwater types is based on very little information on the soil strata underlying the soft organic silt harbor bottom found by the 1954 borings and probes. If actual underlying soil conditions are found to be poor, then the assumptions made for pile capacity and length will be invalid and the results of the design analysis and cost estimate may be significantly different.

REFERENCES

1. Reconnaissance Report - Bridgeport Harbor and Vicinity, Bridgeport, Connecticut - Proposed Navigation Improvements- U.S. Army Corps of Engineers, New England Division, dated April 1980.

2. Minutes of Public Meeting for Navigation Improvement Study at Bridgeport Harbor and Vicinity, Connecticut held at Bridgeport on 19 December 1978.

3. House Document No. 136, 85th Congress, Bridgeport Harbor, Connecticut - U.S. Army Corps of Engineers, New England Division.

4. Shore Protection Manual, Volumes I, II, III - U.S. Army Coastal Engineering Research Center, Corps of Engineers

5. Small Craft Harbors: Design, Construction and Operation Coastal Engineering Research Center, U.S. Army Corps of Engineers, December 1974.

6. Design of Breakwaters and Jetties, Department of the Army, Corps of Engineers, April 1963.

7. Design of Pile Structures and Foundations, Department of the Army, Corps of Engineers, July 1958.

8. Tidal Hydraulics, Department of the Army, Corps of Engineers, August 1965.

9. Floating Breakwaters - State of the Art Literature Review, U. S. Army Engineer Coastal Engineering Research Center, September 1980.

10. Floating Breakwaters Conference Papers, University of Rhode Island, 1974.

11. Brochures for Floating Breakwaters:
    - United Flotation Systems, United McGill Corporation
    - Concrete Flotation Systems, Unifloat Systems Inc.
    - Thompson Floating Breakwater, Thompson Flotation Company

12. Bridgeport Harbor-Black Rock Harbor Soil Boring and Probings Plan and Geologic Section, Corps of Engineers, May 24, 1954.

APPENDIX

CONNECTICUT MARINE SERVICES, INC.
~~1081 Barnum Avenue~~ / Bridgeport, Connecticut ~~06610~~ / (203) 334-0615
One Bostwick Avenue 06605

March 16, 1981

Mr. Benjamin L. Kruser, PE
Vollmer Associates, Inc.
6 St. James Avenue
Boston, Ma. 02116

Dear Mr. Kruser:

As the operators of the City owned Burr Creek Marina, allow me to relate to you the importance of establishing a breakwater at the entrance to Black Rock Harbor.

During the hurricane of last year, a direct southerly wind caused damage estimated to be between $50,000 and $65,000 to docks in the Marina. In addition, loss of revenue per year for the last two seasons is in excess of $100,000. These losses can be directly attributed to excess wave height in the marina during storms because of a lack of protection at the entrance to the harbor.

In addition, twelve boats sank during storms in 1979. The cause of sinkage was attributed to excess wave height in the marina forcing water to overflow into the transoms of the boats thus creating a situation that normal bilge pumps were unable to handle. In a matter of minutes all twelve boats sunk virtually at the same time.

Winds that come out of the South, South East, and South West create extreame tides and unusual wave conditions throughout the harbor. All other area of the harbor are well protected and poise no substantial problems to boatowners and marina operators alike.

Also, when winds are out of the south, many boats break loose from their moorings in the harbor causing our yard crew to take swift precautionary action against runaway boats from damaging our facility and our tenants property. Dangerous situations exist as a direct result of a lack of breakwaters at the entrance to the Harbor.

If you have any questions, please do not hesitate to contact me.

Sincerely,

Dana T. Fiorini
President

cc: Gil Zawadski, Harbormaster

CONNECTICUT MARINE SERVICES, INC.
1081 Barnum Avenue / Bridgeport, Connecticut 06610 / (203) 334-0615

Harbormaster
City of Bridgeport
45 Lyon Terrace
Bridgeport, Connecticut 06604

Dear Gil:

The following is information concerning the urgent need for dredging in the Burr Creek Marina.

TOTAL SLIPS AVAILABLE: 270
TOTAL SLIPS NOW IN USE: 230

Due to the extreame need for dredging, some 40 slips in the marina are now not in use because there is little, or in some cases, no water whatsoever at low tide. In addition, due to the severity of the problem, an additional 25 slips will not be usable next year.

We are presently contemplating closing docks 'G' and 'A' completely next year eliminating approximately 87 slips from the use of boatowmers.

A breakdown of the boats presently using the facilities are as follows:

BOATS UNDER 16':	18
BOATS 16' to 18':	84
BOATS 19' to 21':	40
BOATS 22' to 26':	46
BOATS 27' to 30':	27
BOATS 31' to 37':	15

A breakdown of the average draft of the boats in the marina is as follows:

BOATS UNDER 16':	1' to 1½'
BOATS 16' to 18':	1½' to 2'
BOATS 19' to 21':	2' to 3'
BOATS 22' to 26':	2½' to 3½'
BOATS 27' to 30':	3 to 4½'
BOATS 31' to 37':	3' to 5½'

A breakdown of the average draft of sailboats in the marina is as follows:

4' to 7½' In addition, many of these sailboats are having extreame difficulty operating at low tide. We are afraid that we may have to refuse many of the sailboats now in the marina for next years docking.

The total number of slips that are now not in use due to the severe need for dredging totals 33. In addition, the projection for next year is that an additional 25 slips will be non- useable.

Also, 65 slips in the marina are not being used for the proper size boats due to a severe lack of needed water at low tide.

Virtually every boat in the marina kicks up sludge at low tide, and on a peragee tide, some boats cannot move, while some sailboats actually lean due to a lack of water.

Our estimate indicates, that within four to five years, 90 percent of the marina will be unusable for the average boat.

I hope this information is helpful, Gil. If you have any further questions, please donot hesitate to contact me.

Sincerely,

Dana T. Fiorini
PRESIDENT

Fayerweather Yacht Club, Inc.

51 BREWSTER STREET
BLACK ROCK
BRIDGEPORT, CONN. 06605

2/24/1981

Storm Damage

1980 Damage To Storage Aera Club House and Members Boats (16 Boats lost) $45000.00

1978 Flood Damage and Loss of 12 Boats 40000.00

1977 Flood Damage & Damage To Docks 6000.00

1957 - 10 To 20 Boats Damaged Oct Storm Cost unkown

1955 Club & Dock Damage 20 Boats Damaged or lost

1950 Huricane Damage Boats Lost & Dock Damage cost unkown

1947 Huricane Damage to Club House

1938 Huricane Damage To Club, Docks & Boats Lost

Fayerweather Yacht Club, Inc.

51 BREWSTER STREET
BLACK ROCK
BRIDGEPORT, CONN. 06605

Previous to this time, 1938 very little known.

J P Blackman
P. Commodore F.Y.C.

John H Luby Commander
National Association of
Naval Veterans Port 5
69 Brewster Street
Bridgeport,Conn. 06605
November 6, 1980

Col.William E.Hodgson Division Engineer
Depart ment of The Army
424 Trapelo Road
Waltham,Mass. 02154

Dear Col. Hodgson,

On Nov.3 I spoke to Mr. Don Martin of The Army Engineer Coastal Development Section. He suggested that I foward a letter to your office identifying our organization and the coastal problem we are facing.

The National Association of Naval Veterans Port 5 is a veterans organization. Its 450+ members are from all branches of the Armed Forces. Port 5 is a nonprofit organization dedicated to the betterment of all veterans. Annually Port 5 sponsors trips,fishing contests and outings for the Disabled Veterans of West Haven Veterans Hospital. We are also involved in many community betterment projects.

Port 5 is located in Bridgeport,Conn. Its land and building are directly on Black Rock Harbor (Cedar Creek) which is where our problem is most acute. Port 5s land is at the point where the harbor narrows and therefore acts as a buffer for all other properties further up the harbor. Over the years Port 5 has weathered many storms and they have taken a great toll on our property. Through erosion and partial collapse of the seawall I feel that our building and property are in jeapordy. I believe measures must be taken in the near future to correct this problem or Port 5 will become part of Black Rock Harbor.

This is my reason for seeking the Help of The Army Engineers. Whatever advice or help,if available,you may have in solving this problem would be greatly appreciated.

I realize that I have given only a brief explanation of our problem,but I assure you that it is serious. Therefore,Ias well as all members of Port 5 will be looking foward to your reply.

Respectfully Yours,

John H. Luby

John H. Luby
Commander
National Association of Naval Veterans
Port 5

March 7, 1981

To the Attention of:

Ed Condon
Vollmer Associates

Regarding Black Rock Harbor Storm Damage

Location: 104-118 Seabright Avenue
Black Rock Harbor

Owner: Kaye Williams

Estimated damage incurred from 1969 to 1980

To commercial fishing vessels.....in excess of $ 5,000.
To deck, bulkhead and floats......in excess of $30.000.
To south stone retaining wall.....in excess of $22,000.

Kaye Williams

March 7, 1981

To the Attention of:

Ed Condon
Vollmer Associates

Regarding <u>Black Rock Harbor Storm Damage</u>

Location: Ye Olde Dock Marine School (Middle Wharf)
2 Seabright Avene
Black Rock Harbor

Owner: Margaret Dudas

<u>Estimated damage incurred from 1948 to 1980</u>

To floats, catwalk and gangways.....
To east bulkhead wall.....
To south stone retaining wall........
Total estimated damage to above in excess of $ 74,000.

To members vessels...................in excess of $100,000.

Margaret Dudas

Margaret Dudas

THE BLACK ROCK YACHT CLUB
80 GROVERS AVENUE
BRIDGEPORT, CONN. 06605

March 9, 1981

To: Vollmer Associates, Inc.
Boston, Mass.

From: Peter G. Knight
Fleet Captain

Dear Sirs:

It would be safe to conclude there has been much more storm damage in Blaek Rock Harbor than has been documented here. Every two years, at most, one or more boats meets with misfortune due to heavy seas rolling in the open harbor entrance. The exposed nature of the harbor prevents the further expansion of the anchorage in front of Black Rock Yacht Club by the club or other interests in the Harbor.

We urgently request, as quickly as possible, any help that can be given to protect and aid the development of our beautiful harbor.

Respectfully yours,

Peter G Knight

Peter G. Knight

A PARTIAL HISTORY OF STORM DAMAGE TO

THE BLACK ROCK YACHT CLUB
80 GROVERS AVENUE
BRIDGEPORT, CONN. 06605

prepared by Peter G. Knight February 1981

1893: Aug.24 hurricane destroyed 3 yachts and the pier.

1923: Hurricane wrecked part of the pier, damaged the pool, sank or destroyed several vessels, and ruined the grounds.

1938: Sept. hurricane destroyed deck, dining porch, and entire interior floor. "The club house was nothing but a shell." Boats moored in the harbor were driven ashore where many were battered to pieces.

1949: Another hurricane "not as bad as '38" again damaged the pier, grounds, and moored boats.

1950: A hurricane reduced the club house to a shell again. The sea wall was destroyed and several boats were lost. Damage to club property amounted to $29,000.

1954 &55 Hurricanes destroyed the pier, deck, and several private boats as well as the club race committee boat. Damage to club property was in excess of $20,000.

1961: Hurricane undermined the club. Sidewalks were broken, the pier was damaged, the driveway was washed out and several boats were damaged or destroyed. Damage to club property was in the area of $17,000.

1966: Winter storm caved in dining porch, damaged pier, and washed out the parking lot. Damage in excess of $15,000.

1968: Hurrican eroded fill under the club. The pier and deck were destroyed. The swimming pool and several boats were extensively damaged. Damage to club in excess of $20,000.

1970: Spring hurricane destroyed three boats and broke all floats loose.

1976: Aug. hurricane. Two boats lost $120,000, five more damaged, pool deck damaged, and club house undermined.

1979: Hurricanes destroyed nine boats and damaged five more at a cost of over $300,000. Six pilings were destroyed and the floats and gangway lost or damaged with club cost over $10,000.

1980: Oct. storm. Two boats lost $90,000. Club pier, sea wall, parking lot, dry sailing area, pool deck damaged. Repairs are expected to cost in excess of $20,000.

Regarding damage done by Nov 1950 hurricane

BLACK ROCK YACHT CLUB

Dear Member:

We have now had time to get complete estimates for the reconstruction of the club property and total damage is $29,000. A good bit of the repair will be done by members and should reduce the above figure considerably.

We were most fortunate in that the swimming pool suffered very little damage and will be repaired by members.

To do the various jobs, work parties will be at the club each week end that weather permits. The balance of the work will have to be done by contractors. At the present writing a settlement has not been reached with the insurance companies and until then no definite reconstruction plans can be formulated. A general membership meeting will be called to approve final plans.

One of the biggest problems will be the financing. Prompt payment of dues and house charges will be very helpful and at this time we urge you to send your check for the enclosed dues bill as soon as possible and for the full year.

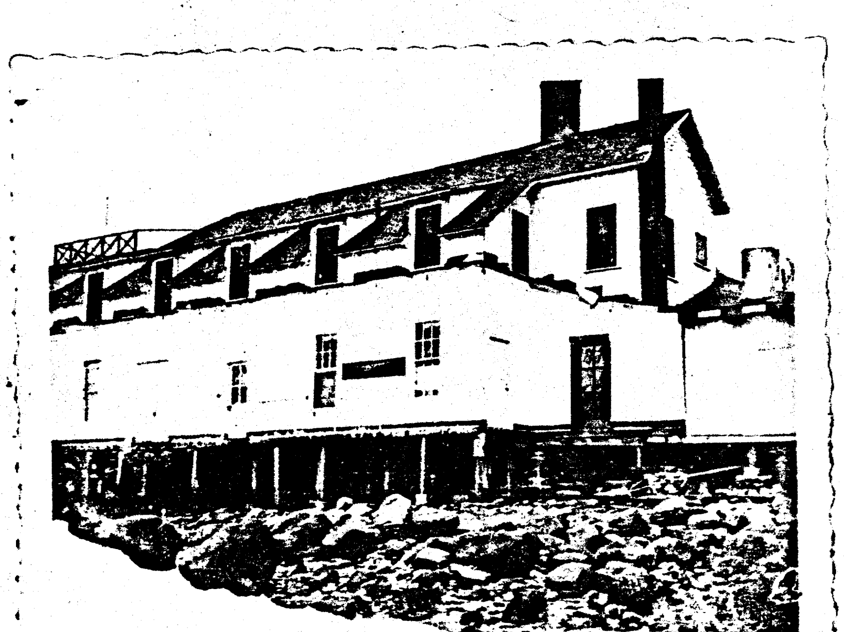
Your officers and governors will be deeply grateful for your cooperation and will keep you informed of our progress.

Sincerely yours,

[illegible signature]

Commodore

1938



BRIDGEPORT SUND

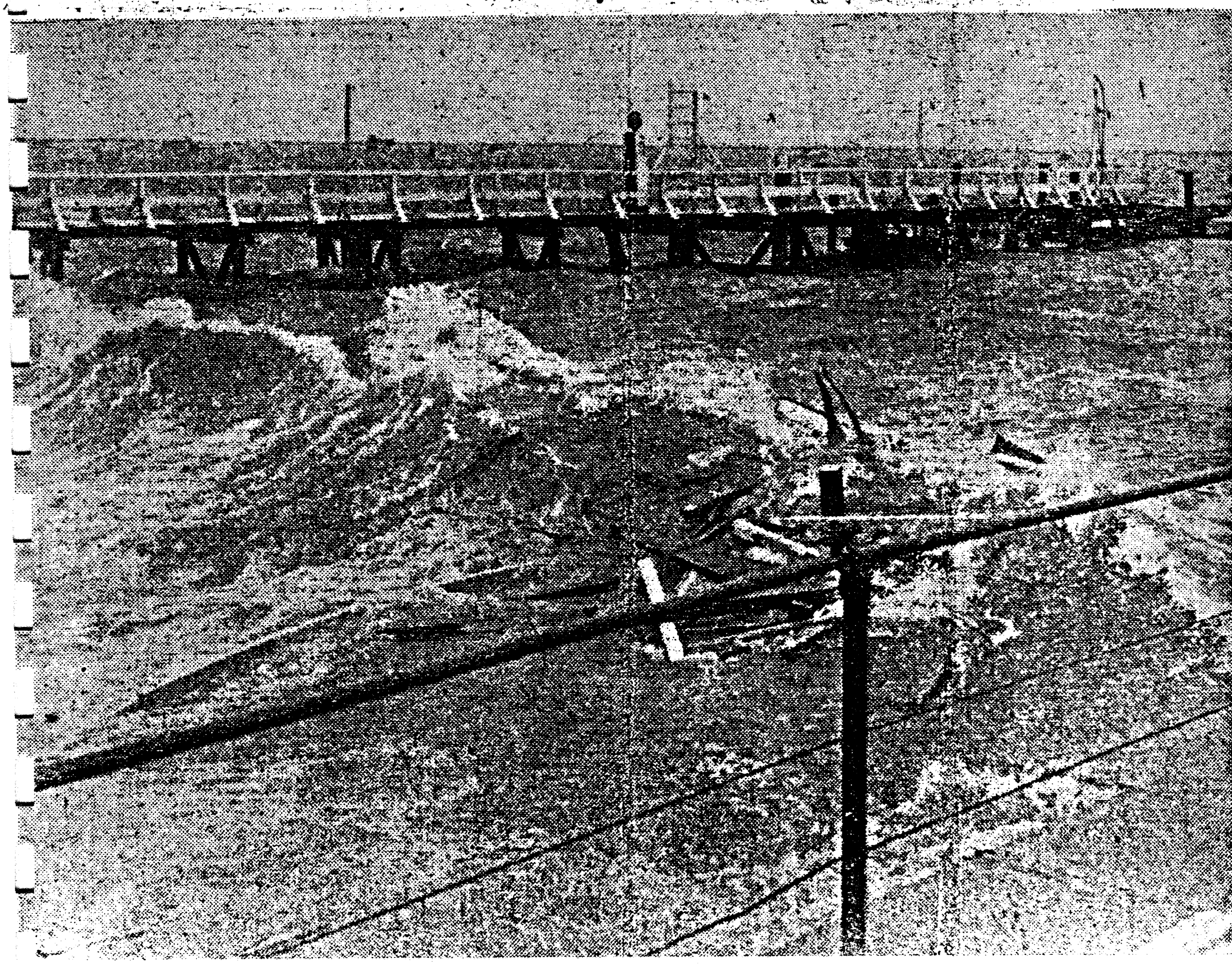
Entered as Second Class Matter.
Post Office, Bridgeport, Conn.

BRIDGEPORT 2, CONN., SUNDAY, OCTOBER 16, 1955

RGENCY DECREED

10/14/55

All That Remains of Black Rock Yacht Club Craft



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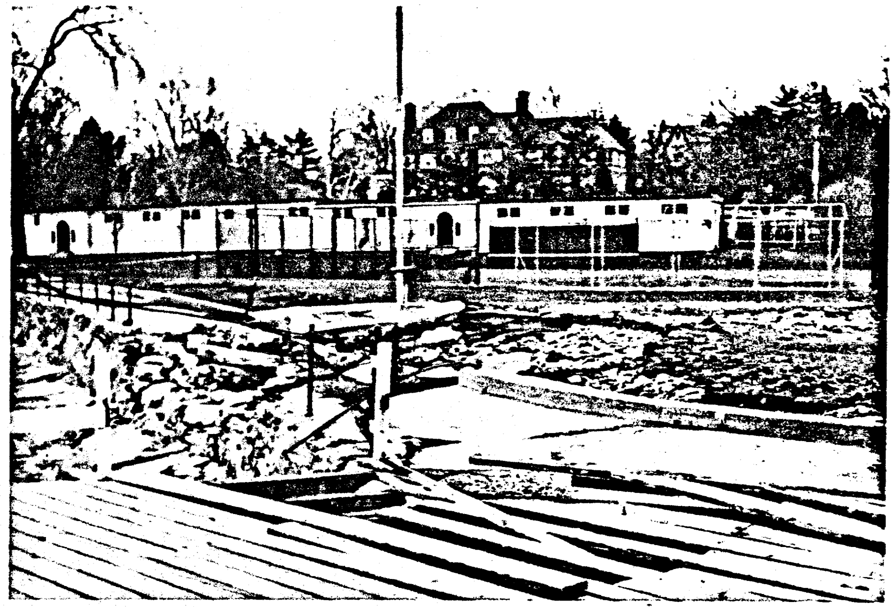

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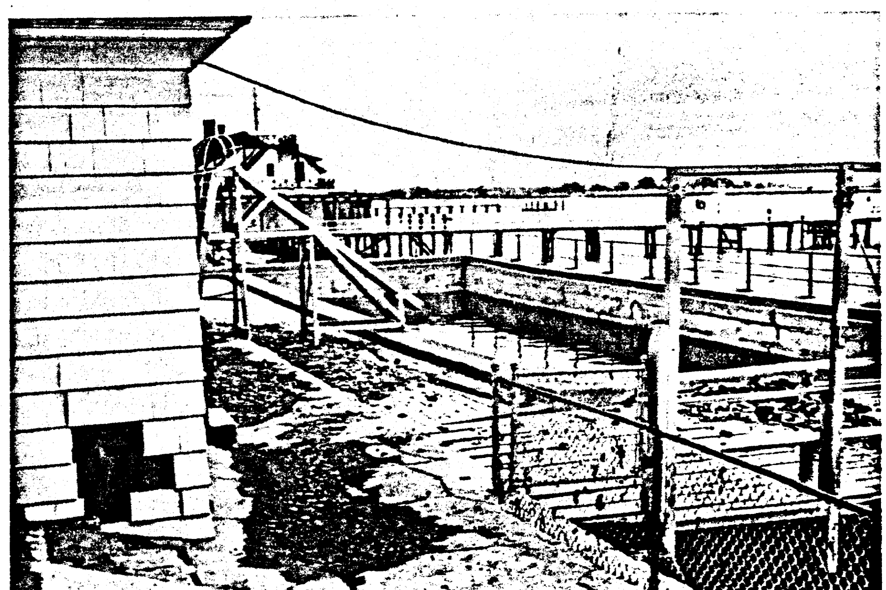

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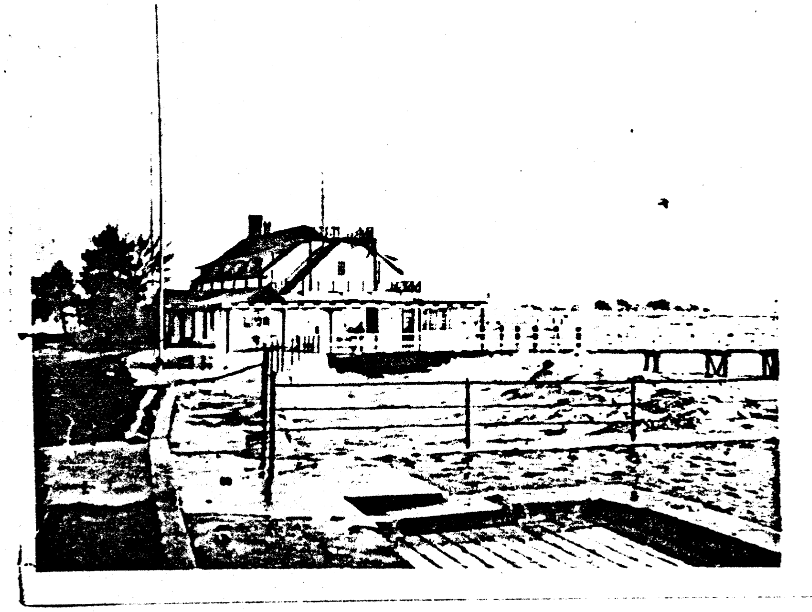

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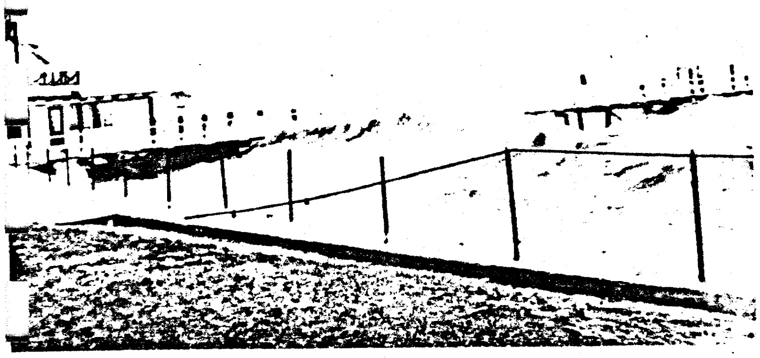

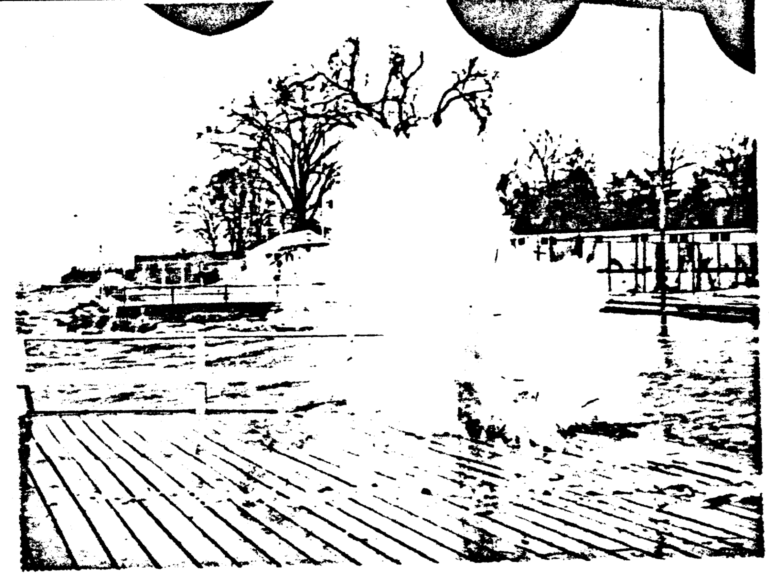

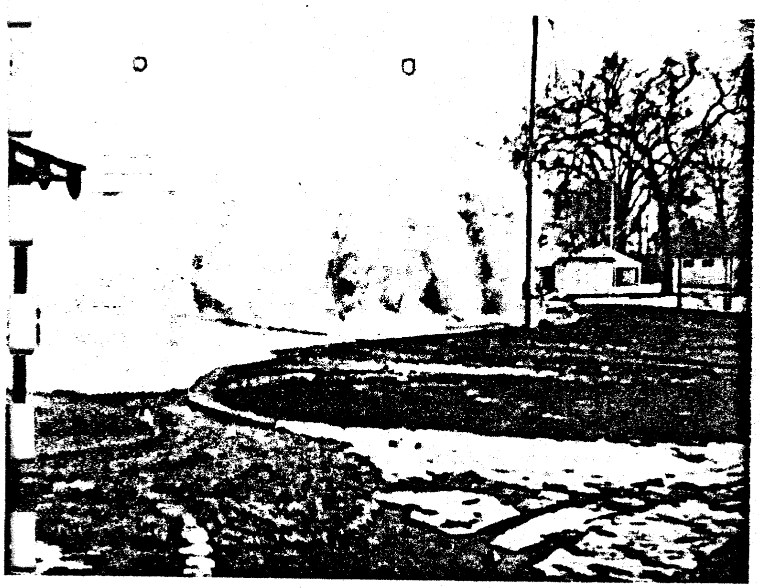

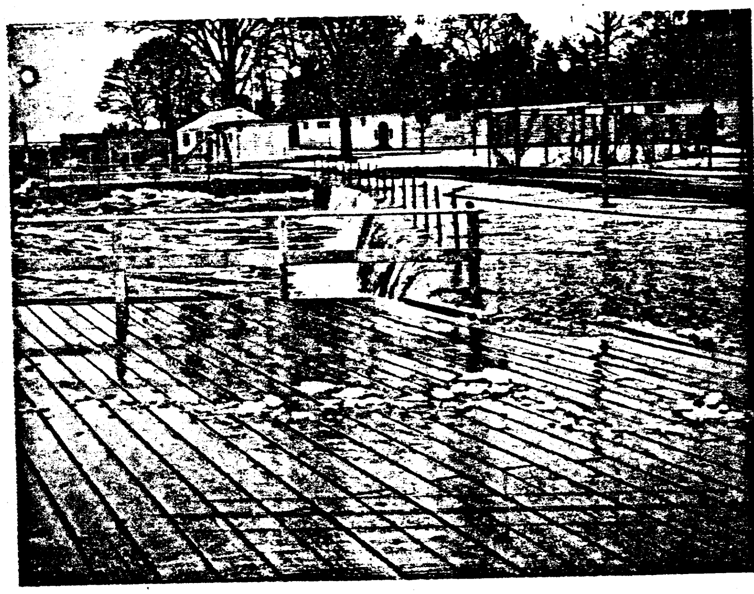

Hurricane "Belle"
8/9-10/76